RICHARD MULLER

HYDROELECTRICAL ENGINEERING

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Where there is will and power:
ask thou no more.
(Dante; Inferno, Canto III.)

THE MISSISSIPPI RIVER POWER DEVELOPMENT AT KEOKUK, IOWA.

Muller, Hydroelectrical Engineering.

HYDROELECTRICAL ENGINEERING

A BOOK FOR HYDRAULIC AND ELECTRICAL ENGINEERS, STUDENTS AND OTHERS INTERESTED IN THE DEVELOPMENT OF HYDROELECTRIC POWER SYSTEMS

BY

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WITH 395 ILLUSTRATIONS IN THE TEXT, 74 TABLES
1 DIAGRAM AND 1 PLATE

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Dedicated

to

Hugh L. Cooper, Esq.

as a token of profound admiration
and affectionate respect

Preface.

The following pages comprise, in a systematic manner, those principles of hydraulic and electrical engineering underlying the design of water power plants. With this end in view and with particular reference to the needs of engineers entrusted with the task of designing and constructing hydroelectric developments or reporting on their commercial success, an earnest effort has been made toward properly selecting and arranging the subject matter.

In a work of this kind, the author has necessarily drawn freely from all sources of information, and he believes that due acknowledgment to them has been made. However, in some instances, search of the original has proved fruitless, and apologies are made to all engineers who may find their work used without any definite reference, such omission being unintentional.

Some of the paragraphs which appear in the chapters on Pressure Pipes and Dams are reprinted with but little alteration from articles contributed by the author to the Engineering Record, Engineering News, and La Technique Moderne.

In many other cases also, items of information have been drawn from the scientific litterature, the chief sources of such information having been, besides the journals mentioned above: The Electrical World, The General Electric Review, The Electrical Journal, The Electrician, Transactions of the American Institute of Electrical Engineers, Transactions of the American Society of Civil Engineers, Zeitschrift des Vereins deutscher Ingenieure, Elektrische Kraftbetriebe und Bahnen. Elektrotechnik und Maschinenbau, Elektrotechnische Zeitschrift, Schweizerische Bauzeitung, Le Génie Civil, La Lumière Electrique, La Houille Blanche, Revue de Mécanique, L'Industrie Minérale, Elettrotecnica, Atti della Associazione Elettrotecnica Italiana.

To the publishers of these journals, special thanks are due for many illustrations from which those of the present volume have been specially prepared.

It is the wish and hope of the author that the collection and unification of the material and the method of treatment are sufficiently complete; the hope is also expressed that this book will be accorded favorable acceptance and consulted with profit.

The author takes this opportunity to express his warm appreciation to Messrs. Hugh L. Cooper and B. H. Parsons, whose advice during several years of hydroelectric work has been invaluable. Thanks are due to Mr. Arthur L. Herrick, C. E., for his careful reading of the proofs and for the many important suggestions which he has made; to the publisher, for the special care and effort taken in accomplishing the publication of this book.

New York, August 1921.

Richard Muller.

Contents.

Chapter I.

Hydrology.

Chapter II.

Stream Measurements.

Chapter III.

Canals.

Chapter IV.

Pressure Pipes.

Chapter V.

Dams.

Chapter VI.

Turbines.

Chapter VII.

Power House and Substation Equipment.

Chapter VIII.

Transmission Lines.

Chapter IX.

Investigation of Water Power Projects. Economics. Page

Chapter X.

Description of Hydroelectric Plants.

Appendix.

Evaluation of Water Powers. Legislation.

Chapter I. Hydrology.

1. Classification of Streams.

Streams may be classified according to the characteristics of their physical aspect, or otherwise, according to the relation of rainfall to run-off in their hydrological basin.

In the first case four classes may be established:

1) *Large rivers*, or streams of considerable discharge, small slope and nearly constant height of water level, except when torrential rains occur, then the flood discharge is considerable and prolonged.

2) *Torrential rivers*. These generally have their origin in mountainous regions and may be considered as being the tributaries of large rivers. Torrential rivers are characterized as flowing through narrow gorges of steep slopes, and are subject to sudden fluctuations due to excessive rainfall in their catchment area.

3) *Torrents* are torrential rivers of short length, much steeper, and carrying stones and much silt.

4) *Brooks or creeks* consist of small streams running slowly through flat country. Their water is generally clear, as the velocity is too low to wash the bed.

Rivers may also be classified as regards run-off, that is by their "family" resemblances [1].

"In the first class will fall streams where the maximum rainfall is from 50 to 60 inches, with corresponding run-off somewhat more than one half of the rainfall. The minimum run-off will be about one half the rainfall or a little less. These statements are general ones to which there are exceptions. Another class of streams are those with maximum rainfall on their catchments of 40 to 50 inches and with corresponding run-off somewhat less than one half the rainfall. The minimum run-off for these streams is from one sixth to one fourth of the corresponding rainfall, or from about 16 to 25%. A further class, the far Western streams, may be mentioned, in which the run-off is only a very small percentage of the rainfall, in some cases not more than 4 to 5%, or at times even less.

Probably, comprehensive study would further subdivide these streams, but the intention at present is to merely call attention to some of the more marked peculiarities as a basis for final detailed study.

In addition to the classification of streams with reference to rainfall and run off, those with large evaporations may be placed in a class by themselves. Streams with large evaporation are, so far as known, always deforested."

The rivers which are considered as best adaptable to the development of water power are those that offer a large discharge, a steep slope and a long course. These ideal conditions are seldom to be found however.

[1] Relation of Rainfall to Run-off. U. S. Geological Survey Water Supply and Irrigation Paper No. 80.

2. Rainfall.

All the water flowing in rivers is precipitated from the atmosphere as rain or snow. A warm atmosphere has a larger capacity for holding moisture, but on cooling, a temperature will be reached where the water begins to condense forming vapor or "clouds". The atmosphere is then said to be saturated, or to possess a relative humidity of one hundred per cent. If the process of condensation is rapid rainfall will occur.

The quantity of yearly rainfall in a drainage basin is difficult to evaluate exactly, although its intensity and distribution are the factors which most influence the run-off.

Such amounts of rainfall may vary at different times for a same locality, at the same time for different locations, and between great limits during a period of one year. The mean yearly and seasonal rainfall for a same locality is nearly constant, as has been observed from records extending over several years.

It has been noted generally, that the amount of rain increases with the altitude, although this is not absolutely true for all cases.

This amount is largest in the proximity of the ocean or of large bodies of water and along mountain ranges, especially but not necessarily if the range is parallel to the sea coast, close to it and with its direction at right angle to the direction of prevailing winds. At higher altitudes, rapid cooling of the moisture laden air happens, due to the expansion of the elastic air, and first condensation, then precipitation takes place. On the lee-side of the mountains, when the winds descend again to the low-lands, the atmosphere is often arid.

Rain gauges or pluviometers are instruments of very simple design, destined to receive and measure the amount of rainfall for a locality where such instruments are placed.

The pluviometer used by the United States Weather Bureau is well suited for the required measurements under ordinary conditions. This instrument consists of a receiver, an overflow attachment, and a measuring tube. The exposed surface is connected by means of a reducing funnel with the measuring tube, which has an inside area of cross section of $^1{}_{10}$ the area of the surface of the receiver. Therefore the depth of precipitation is ten times less than the measured depth of water in the tube.

The registering gauges, which are the most interesting ones, record at the same time the intensity as well as the length of time of the rainfall. Inaccuracies in rainfall records are sometimes due to poor exposure of instruments, or incorrect measurements of snowfall. It has been observed in most cases of incorrect recording, that the wind is more or less responsible, and such inaccuracy will vary directly with the intensity of the wind.

A rain gauge should preferably be placed in an open lot, disposed so that its superior orifice be in a horizontal plane, unobstructed by large trees or buildings. Nevertheless, there is a part of the precipitation which escapes the rain gauges placed in the drainage areas formed by very high mountains: this is the condensation which takes place on their summits at very high altitudes.

Snowfall is measured with more or less difficulty, due to the fact that it is a difficult task to collect in a receptacle the real amount of snow

falling when there is wind. The best way, it is thought, is to allow the snow to fall on a prearranged platform, determine the definite area, collect a sample of it and melt it so that its water equivalent may be measured.

Considering that observation stations are located in the lower altitudes, and that they do not possibly represent the records of precipitations at higher altitudes, it is seen that the application of a few records to a large area may lead to considerable error. The horizontal and vertical distribution of stations is usualy made as uniform as possible and in such locations as to be readily accessible to a competent observer; when importance of locality warrants it, such stations are in telegraphic proximity, and excessive rains are reported.

Comparison of rainfall with measured stream flow shows that great errors will undoubtedly result, in attempts to determine the amount of water available or under consideration from rainfall data. In using such recorded data, the conditions affecting its accuracy must be considered. It should be borne in mind that such records show only precipitation for small specified areas which are not representative of a considerable area. The number of gauges in any particular drainage basin must therefore be large.

In order to compare rainfall and run-off, both records should be expressed in depths in inches for the same drainage area under consideration, and should pertain to the same periods of time. As such observations, recorded through a few months are of no practical use, they must extend as much as possible over a period of one or more years.

"Rainfall, as is well known, varies in amount in different parts of the world from almost nothing in the deserts of Africa, Central Asia, and the high lands of Peru, up to as much as 600 inches per annum at Cherrapongee in the Himalayas, and even within much narrower limits its variation in amount is very marked; for instance in England it varies from about 20 inches in some of the eastern counties up to 140 to 165 inches per annum in the hills of Cumberland, Wales and Scotland[1]".

R. F. Teele[2] states that the annual rainfall of eastern Minnesota, Wisconsin and Michigan ranges from 30 to 45 inches... Typical of the North Atlantic states, the normal annual rainfall of New Jersey is approximately 45 inches... The most humid portion of the agricultural East is subject to the greatest irregularity of rainfall. Mr. Teele refers to the southern states bordering on the Gulf of Mexico and the Atlantic Ocean, where the annual precipitation ranges from 45 to 55 inches.

3. Run-off.

Of the water which falls on the basin of a stream, that part which is not taken into the organism of plants, lost by evaporation of the sun, or absorbed by the earth, finds its way into streams as surface flow or run-off.

M. Vermeule says that "Stream flow includes the water which passes directly over the surface to the stream, and also that which is temporarily absorbed by the earth to be slowly discharged into the streams. A portion, usually extremely small, passes downward into the earth and appears neither as evaporation nor as stream flow. It is usually too small to be considered, and we may for our purpose assume that all of the rain which falls upon a given watershed and does not go off as stream flow is evaporated, using the latter word in the broadened sense."

The rainfall being expressed in inches in depth, it has been found convenient, for comparative purposes, to express the run-off likewise, that

[1] Sir Alexander R. Binnie, in Rainfall Reservoirs and Water Supply. Page 4.

[2] R. P. Teele: Irrigation in the United States.

is to say in inches in depth over the entire water shed. The rate of flow, computed for power purposes, is always expressed in cubic feet per second.

4. Mean Monthly Rainfall.

Particular importance must be placed on the monthly distribution of rainfall. It is an observed fact that rain is more abundant in the winter

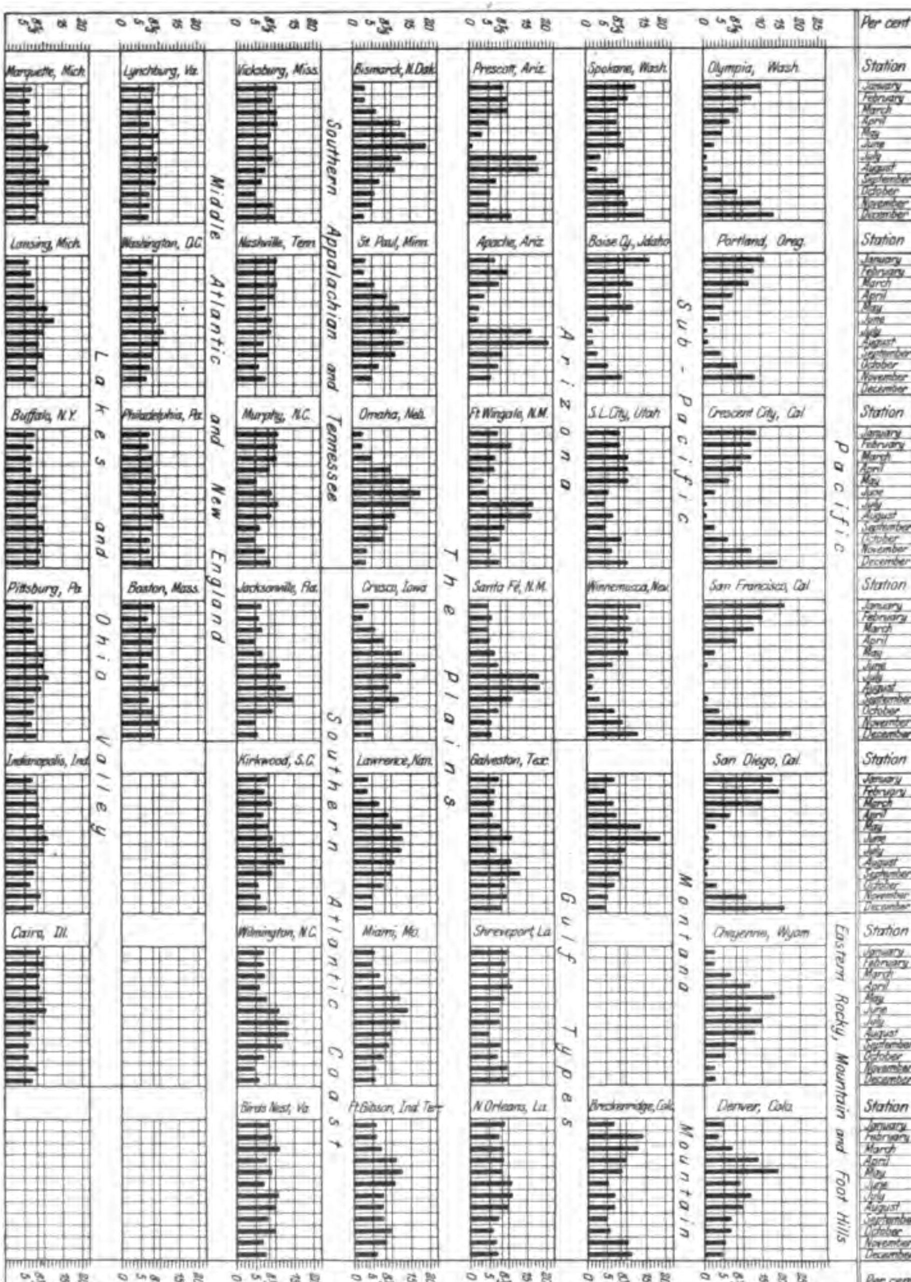


Fig. 1. Types of Monthly Distribution of Precipitation in the United States.
Rainfall Distribution in the U. S. (Percentage of fall in each month represented by heavy lines.)

months than in summer, when evaporation is at a maximum and vegetation absorption is greatest. Such monthly flows do have an immediate bearing on the necessary storage capacity of reservoirs.

Fig. 1 shows graphically the typical average monthly distribution of different sections of the country. The abscissae represent the percentage of the total yearly rain falling in the month.

The monthly distribution varies in different localities although there is some similarity in distribution if some similarity in conditions exists.

For instance, in the Middle Atlantic and New England States, the distribution is quite uniform. In the plains a large percentage of the rain is falling in the summer months, and the distribution along the entire Pacific Coast is very similar to that at San Francisco, California.

5. Extended Rainfall Records.

According to Mr. A. R. Binnie[1], dependence can be placed on any good record of 35 years' duration to give a mean rainfall correct within 2% of the truth.

G. W. Rafter states that for records extending over a period of 20 to 35 years in length the error may be expected to vary from 3.25 to 2.00% and that for shorter periods of 5, 10 and 15 years, the probable extreme deviation from the mean would be 15, 8.25, and 4.75% respectively. A 20 years record, therefore, may be expected to show an error of 3.24%. This is about as close as rainfall records in this country can be expected to agree, as comparatively few are much beyond 20 years in extension.

In his paper on the rainfall in the United States, Mr. Henry has examined this question, using long records at New Bedford, St. Louis, Philadelphia, Cincinnati, and other places. The rainfall has been measured at New Bedford for 83 consecutive years, and at St. Louis for 60 years. For a 10 year period Mr. Henry found the following variations from the normal:

at New Bedford	+ 16%	and	— 11%
„ Cincinnati	+ 20%	„	— 17%
„ St. Louis	+ 17%	„	— 13%
„ Forth Leavenworth	+ 16%	„	— 18%
„ San Francisco	+ 9%	„	— 10%

For a 25 year period it was found that the extreme variation was 10% both at St. Louis and New Bedford. Mr. Henry reached the conclusion that at least 35 to 40 years' observations are required to obtain a result that will not depart more than $\pm$ 5% from the true normal. The average variation of a 35 year period was found to be $\pm$ 5% and for a 40 year period $\pm$ 3%.

6. Relation of Rainfall to Run-off.

Although it is generally impracticable to determine the quantity of run-off from the amount of rainfall, it has been observed that there is a more or less direct relation between run-off of streams and rainfall over their drainage area. The observed run-off however, does not vary in direct

[1] On Mean or Average Rainfall and the Fluctuations to which it is subject, by Alexander R. Binnie. Inst. C. E.: Proc. C. E. Vol. CIX pp. 89—172.

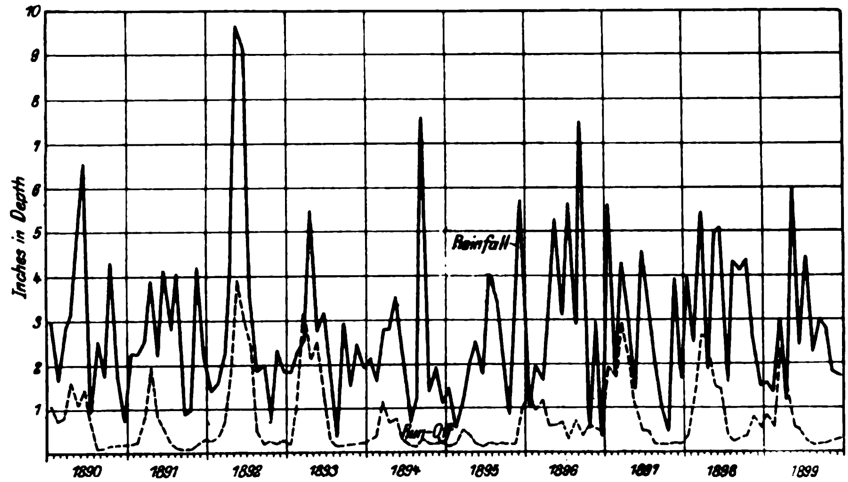


Fig. 2. Comparison of Mean Monthly Rainfall and Run-off of Illinois River Basin.

proportion to the rainfall. For instance Fig. 2, which is a comparative diagram of monthly rainfall and run-off of the Illinois River basin, shows for the year 1896, that to an increase of rainfall does not correspond an increase in run-off. Although heavy raining has occured in the summer months of that year, the diagram seems to show that percolation and evaporation must have been very active.

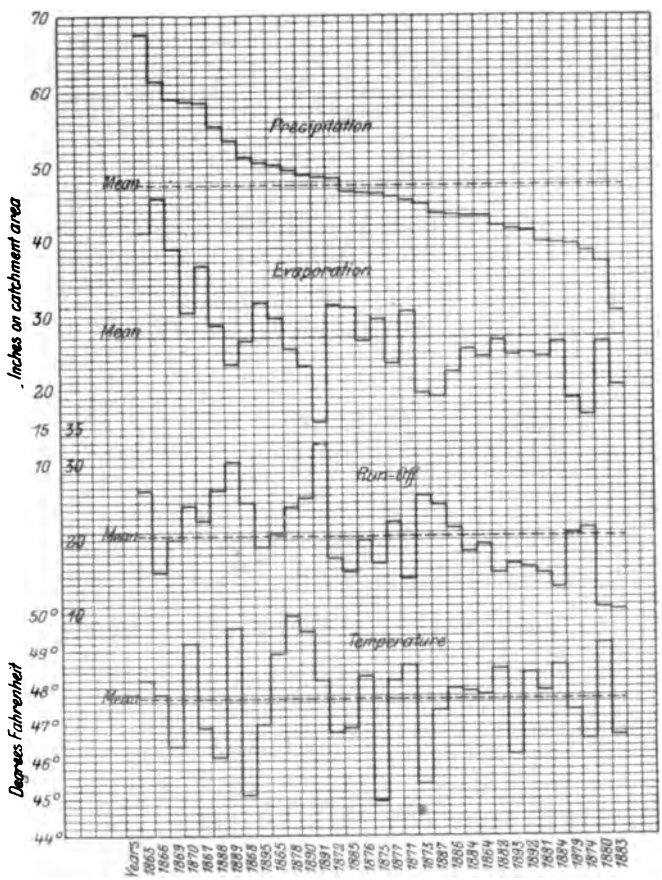


Fig. 3.

Fig. 3, shows for Lake Cochituate, precipitation, evaporation, run-off and mean annual temperature, plotted in the order of the precipitation.

Mr. Geo. W. Rafter[1] has made a careful analysis of the available data, and some of his conclusions are as follows:

a) There is no general expression giving accurately the relation of rainfall to run-off. The run-off of a stream is influenced by so many

[1] The Relation of Rainfall to Run-off. Water supply and Irrigation Paper No. 80.

complex elements that the data are lacking for final conclusions. Every stream is in effect a law unto itself. An empirical formula may, however, be made, which will give for some streams approximately the run-off for a series of years.

b) The run-off of streams has generally been overestimated. The minimum flow may be as low as from 0.05 to 0.1 of a cu. ft. per second per square mile. Streams issuing from sand plains may show from 0.5 to 0.6 of a cubic foot per square mile per sec. Generally speaking the range will not be outside of 0.05 to 0.5 of a cubic foot per square mile per second.

c) The run-off of a stream is materially influenced by the number of lakes within its catchment area.

d) Safe deductions can not be made from an average run-off. What is wanted is a clear statement of the minimum, together with the longest period which it may be expected to occupy.

e) The influence of the May rainfall is such that when above the normal, stream flow is likely to be well maintained during the summer.

f) When rainfall is below the mean for several months, ground water may be expected to become continuously lower, with the result that the flow of the stream will be less.

g) The run-off of streams will vary with the velocity of wind, pressure, force of vapor, etc.

h) When a run-off record is given, without the rainfall, the rainfall may be computed by assuming the evaporation and making a series of approximations.

i) The run-off of streams with very great difference in size of catchment areas may be experimentally compared.

j) The extreme low water period may extend over at least two years and occasionally over three years.

k) The ground water must be taken into account in order to understand all the peculiarities of flow. A very important effect of forests is in increasing the ground water flow.

l) It is uncertain whether difference in geology has much influence as to run-off, although it appears that porous sandy soils do considerably affect the result. There are, however a number of cases which indicate that it may have important influence, although an examination of the evidence shows that the theory that forests materially influence the run-off is more reasonable than that percolation through geologic formations exercises much influence.

7. The Water Year.

Rainfall and run-off records are conveniently divided into three periods, for the greater portion of the United States, in order to permit a comprehensive understanding of their relation. These periods begin on December 1st and end Nov. 30th.

The storage period includes the months from December to May inclusive. During this period, the evaporation and absorption by the organisms of plants are relatively slight, and it follows to a great degree that the amount of water which can be stored is exhibited by the rainfall of the storage months.

The growing period extends over the months of June, July and August, and these being the summer months, evaporation and plant absorption are at a maximum. M. Rafter states that frequently not more than 0.1 of the rainfall appears in the streams, and sometimes not more than 0.05 or even less.

The replenishing period includes the months of September, October and November. In these months the rainfall is normal and the run-off is larger than during the growing period.

An analysis of such data for the Muskingum, Hudson and Connecticut River as made by M. Rafter, are shown in tables I, II, III, and table IV shows the mean or average rainfall run-off and evaporation for storage, growing and replenishing periods for 12 streams of this country.

Table I. *Muskingum River, 1888—1895, inclusive.*

[Catchment area = 5,828 square miles.]

Period.	1888.			1889.			1890.		
	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.
Storage	17.16	5.17	11.99	13.52	6.02	7.50	27.77	18.07	9.70
Growing	14.31	1.77	12.54	12.12	1.24	10.88	13.68	2.64	11.04
Replenishing	11.14	3.39	7.75	10.24	0.96	9.28	15.52	6.13	9.39
Year	42.61	10.33	32.28	35.88	8.22	27.66	56.97	26.84	30.13
	1891.			1892.			1893.		
Storage	16.72	12.42	4.30	20.39	9.06	11.33	25.04	14.13	10.91
Growing	13.56	1.77	11.79	16.54	3.65	12.89	8.31	1.22	7.09
Replenishing	7.08	1.37	5.71	4.81	0.67	4.14	9.01	0.85	8.16
Year	37.36	15.56	21.80	41.74	13.38	28.36	42.36	16.20	26.16
				1894.			1895.		
Storage				16.93	7.63	9.30	13.04	4.04	9.00
Growing				4.56	0.66	3.90	9.14	0.49	8.65
Replenishing				9.02	0.41	8.61	7.66	0.37	7.29
Year				30.51	8.70	21.81	29.84	4.90	24.94

Table II. *Hudson River, 1888—1901, inclusive.*

[Catchment area = 4,500 square miles.]

Period.	1888.			1889.			1890.		
	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.
Storage	20.40	17.06	3.34	17.10	14.04	3.06	24.75	19.28	5.47
Growing	10.25	2.05	8.20	15.05	4.26	10.79	13.50	2.85	10.65
Replenishing	13.27	4.53	8.74	10.81	3.41	7.40	12.10	6.81	5.29
Year	43.92	[a]23.64	20.28	[a]42.96	21.71	21.25	[a]50.35	28.94	21.41

[a] Approximate.

Period.	1891.			1892.			1893.		
	Rain-fall	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.
Storage	20.69	16.59	4.10	24.95	22.50	2.45	19.83	15.20	4.63
Growing	13.49	2.07	11.42	19.12	6.87	12.25	13.37	3.12	10.25
Replenishing	8.78	1.90	6.88	9.80	3.71	6.09	8.98	3.59	5.39
Year	42.96	20.56	22.40	53.87	33.08	20.79	42.18	21.91	20.27
	1894.			1895.			1896.		
Storage	21.37	13.18	8.19	15.79	11.68	4.11	22.17	16.52	5.65
Growing	8.73	3.20	5.53	10.37	2.36	8.01	10.25	2.53	7.72
Replenishing	11.87	2.99	8.88	10.51	3.42	7.09	12.79	4.58	8.21
Year	41.97	19.37	22.60	36.67	17.46	19.21	45.21	23.62	21.58
	1897.			1898.			1899.		
Storage	19.77	14.60	5.17	22.80	18.61	4.19	19.48	15.15	4.33
Growing	15.80	7.79	8.01	13.52	3.24	10.28	7.40	1.63	5.77
Replenishing	10.94	3.80	7.14	12.19	5.27	6.92	8.91	2.76	6.15
Year	46.51	26.19	20.32	48.51	27.12	21.39	35.79	19.54	16.25
				1900.			1901.		
Storage				21.13	16.12	5.01	18.47	14.84	3.63
Growing				12.11	2.30	9.81	15.09	4.02	11.07
Replenishing				12.17	2.25	9.92	9.02	3.00	6.02
Year				45.41	20.67	24.74	42.58	21.86	20.72

Table III. *Connecticut River, 1872—1885, inclusive.*

[Catchment area = 10,234 square miles.]

Period.	1872.			1873.*			1874.*		
	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.
Storage	14.92	13.30	1.62	18.16	21.80	−3.64	23.08	23.04	0.04
Growing	18.96	6.29	12.67	10.11	2.71	7.40	14.37	6.62	7.75
Replenishing	12.42	6.64	5.78	15.04	5.22	9.82	7.76	2.15	5.61
Year	46.30	26.23	20.07	43.31	29.73	13.58	45.21	31.81	13.40
Period.	1875.			1876.*			1877.		
Storage	17.51	15.47	2.04	22.50	24.74	−2.24	18.09	12.68	5.41
Growing	14.55	3.80	10.75	12.51	3.35	9.16	14.00	2.91	11.09
Replenishing	11.36	3.60	7.76	10.57	2.28	8.29	13.08	5.27	7.81
Year	43.42	22.87	20.55	45.58	30.37	15.21	45.17	20.86	24.31
Period.	1878.			1879.			1880.		
Storage	21.88	18.02	3.86	23.19	21.49	1.70	18.29	14.78	3.51
Growing	13.59	3.45	10.14	16.07	2.92	13.15	11.82	2.45	9.37
Replenishing	10.56	3.06	7.50	9.48	2.93	6.55	11.58	2.62	8.96
Year	46.03	24.53	21.50	48.74	27.34	21.40	41.69	19.85	21.84

* Not included in mean.

Period.	1881.			1882.			1883.		
	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.	Rain-fall.	Run-off.	Evapo-ration.
Storage	20.83	16.02	4.81	[a]20.50	12.14	8.36	[a]12.85	8.73	4.12
Growing	11.30	2.93	8.37	[a]11.45	3.35	8.10	[a]13.50	2.51	10.99
Replenishing	11.38	3.39	7.99	[a]6.50	2.17	4.33	[a]6.20	1.37	4.83
Year	43.51	22.34	21.17	38.45	17.66	20.79	32.55	12.61	19.94

Period.	1884.			1885.		
Storage	21.42	20.20	1.22	18.58	13.63	4.95
Growing	12.14	2.79	9.35	14.82	3.20	11.62
Replenishing	8.51	2.61	5.90	11.76	5.61	6.15
Year	42.07	25.60	16.47	45.16	22.44	22.72

[a] Rainfall computed, approximate.

Table IV. *Mean or average rainfall, run-off, and evaporation for storage, growing, and replenishing periods for 12 streams of the United States.*

Period.	Muskingum River, from 1888 to 1895, eight years. Catchment area, 5,828 square miles.			Genesee River, from 1890 to 1898, nine years. Catchment area, 1,070 square miles.			Croton River, from 1877 to 1899, twenty-three years. Catchment area, 338,8 square miles.		
	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.
Storage	18.8	9.6	9.2	19.4	10.5	8.9	23.7	16.8	6.9
Growing	11.6	1.7	9.9	11.5	1.7	9.8	13.6	2.6	11.0
Replenishing	9.3	1.8	7.5	9.4	2.0	7.4	12.1	3.4	8.7
Year	39.7	13.1	26.6	40.3	14.2	26.1	49.4	22.8	26.6
Period.	Lake Cochituate, from 1863 to 1900, thirty-eight years. Catchment area, 18.9 square miles.			Sudbury River, from 1875 to 1900, twenty-six years. Catchment area, 78.2 square miles.			Mystic Lake, from 1878 to 1895, eighteen years. Catchment area, 26.9 square miles.		
	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.
Storage	23.1	14.9	8.2	23.5	17.9	5.6	22.4	15.1	7.3
Growing	11.6	2.1	9.5	10.7	1.7	9.0	10.9	2.3	8.6
Replenishing	12.4	3.3	9.1	11.9	3.0	8.9	10.8	2.6	8.2
Year	47.1	20.3	26.8	46.1	22.6	23.5	41.1	20.0	24.1
Period.	Neshaminy Creek, from 1884 to 1899, sixteen years. Catchment area, 139.3 square miles.			Perkiomen Creek, from 1884 to 1899, sixteen years. Catchment area, 152 square miles.			Tohickon Creek, from 1884 to 1898, fifteen years. Catchment area, 102.2 square miles.		
	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.
Storage	23.1	17.2	5.9	23.2	16.7	6.5	24.2	20.5	3.7
Growing	13.4	2.7	10.7	13.7	3.1	10.6	14.6	3.5	11.1
Replenishing	11.1	3.2	7.9	11.1	3.8	7.3	11.3	4.4	6.9
Year	47.6	23.1	24.5	48.0	23.6	24.4	50.1	28.4	21.7
Period.	Hudson River, from 1888 to 1901, fourteen years. Catchment area, 4,500 square miles.			Pequannock River, from 1891 to 1899, nine years. Catchment area, 63.7 square miles.			Connecticut River, from 1872 to 1885, eleven years.[b] Catchment area, 10,234 square miles.		
	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.	Rain.	Run-off.	Evapo-ration.
Storage	20.6	16.1	4.5	23.0	19.7	3.3	18.9	15.1	3.8
Growing	12.7	3.5	9.2	12.7	3.1	9.6	13.8	3.3	10.5
Replenishing	10.9	3.7	7.2	11.1	4.0	7.1	10.3	3.6	6.7
Year	44.2	23.3	20.9	46.8	26.8	20.0	43.0	22.0	21.0

[b] Three years omitted from mean.

8. Evaporation.

The subject of evaporation is one of much interest; its estimate is the measurement of actual and immediate loss of water and power, measurement which involves many difficulties.

Of the water in the form of rain, one part is returned to the sea by the riverways, the other part returns to the atmosphere through successive evaporations. Such evaporation occurs as well at the surface of the water as on land, the process being nearly continuous in the first case and intermittent in the other. The amount of evaporation depends on many factors, all of which, although involving complications, must be well understood and estimated, and in effecting such measurements, it must be borne in mind that the process of evaporation is variable with respect to time and locality, or in other words, it varies like rainfall, in different localities at the same time and in the same locality at different times.

In the study of artificial storage, great value is attributed to the knowledge of the relative amount of water returned to the atmosphere by evaporation, since evaporation is the amount of annual rainfall diminished by the amount of annual run-off.

The relations of evaporation as established by Prof. Thomas Tate which are of interest to water power engineers are as follows:

a) Other things being the same, the rate of evaporation is nearly proportional to the difference of the temperature indicated by the wet-bulb and dry-bulb thermometers.

b) Other things being the same, the augmentation of evaporation due to air in motion is nearly proportional to the velocity of the wind.

c) Other things being the same, the evaporation is nearly inversely proportional to the pressure of the atmosphere.

d) The rate of evaporation from different substances mainly depends upon the roughness of, or inequalities on, their surfaces, the evaporation taking place most rapidly on the roughest or most uneven surfaces; in fact, the best radiators are the best vaporizers of moisture.

Evaporation as has been said above, is the amount of annual rainfall diminished by the amount of annual run-off; algebraically expressed:

$$E = R - F$$

For an approximate estimate of evaporation, Mr. FitzGerald's formula may be used. Calling

$V =$ the maximum force of vapor in inches of mercury corresponding to the temperature of the water;
$W =$ the velocity of the wind in miles per hour;
$E =$ the evaporation in inches of depth per hour;
$v =$ the force of vapor present in the air;

said formula has the form:

$$E = \frac{(V - v)\left(1 + \frac{W}{2}\right)}{60}$$ [1]

[1] Transactions Am. Soc. C. E. Vol. XV pp. 581—646.

The force of vapor present in the air is, however, dependent on the temperature of the air and the height of the barometer, so that the value of v in the foregoing formula must be computed beforehand from the following relation:

$$v = v_1 - \frac{0.48\,(t - t')}{689 - t'}\,h,$$

in which:

v = the force of vapor in the air at the time of the observation;
v_1 = the force of vapor in a saturated air at the temperature t';
t = the temperature of the air in degrees centigrade, indicated by the dry thermometer;
t' = the temperature of evaporation given by the wet thermometer;
h = the height of the barometer.

The results of evaporation show clearly that the constituents of water are continuously changing state, from gaseous to liquid and vice versa.

The rate of conversion from water to vapor is at its minimum when the atmosphere is humid and still, not being agitated by winds of important movement, but the process of evaporation increases in rapidity when the atmosphere is drier and hotter, when the movement of wind is considerable and when the water itself is somewhat warm.

Evaporation is at its maximum at the equator and gradually diminishes towards the poles. It has been observed that vegetation increases evaporation, and that most summer rains evaporate nearly entirely, while winter precipitation remains longer on the surface in the form of snow, converting itself into spring flow mostly, and later in summer runs off from underground nappes.

Observation of evaporation may be satisfactorily made by means of boxes sunk in the earth and about flush with, the surface of the ground, or floated in streams or ponds. The results of such observations should contain remarks as to conditions of wind, temperature of air, water inside and outside of box, and amount of precipitation.

9. Estimate of Stream Flow and Power.

To measure the discharge of a river involves a series of delicate observations, the accuracy of which depends upon the care brought in the measurements. It is well to bear in mind, inasmuch as the flow of a stream is always changing, that measurements and data of reasonable accuracy, extending over a period of several years, will be of more importance than measurements and data taken over a relatively short period.

For instance, as regards velocity measurements, should observations be made in time of maximum flow for a particular river, the average velocity will be too large, whereas if they are made during a low water period, the average velocity will be too small. Therefore, it is of vital importance to gauge specified rivers during all seasons of the year, so as not only to cover their total discharge, but also their seasonal distribution.

The influence of precise measurements on determination of available power is shown in the following. The flow of a stream is a continuous process and its power can be stated in terms of horse-power hours. One continuous horse-power for 24 hours per day and 365 days in the year represents 8760 horse-power hours.

In practice, it is usually estimated that power will be used for 10 to 16 hours per day during 310 working days. When time is not expressed, it is understood that 24-hr 365-day power is meant. Therefore, in a working year, each horse-power capacity represents from 3100 to 4960 horse-power hours. Now there is a difference of (8760—3100) and (8760—4960) or 5560 and 3800 horse-power hours which could be stored, and the capacity of the plant per hour for the working time would be

$$\frac{8760}{3100} = 2.83 \text{ and } \frac{8760}{4960} = 1.76 \text{ H. P.}$$

for the average horse-power of the stream.

In other words, a plant working for 10 hours a day and 310 days in a year could deliver 2.83 H. P. per hour for each horse power capacity of the stream if the stream flow can be stored during the remaining 14 hours a day and the remaining 55 days. In the second case, the delivery would be 1.76 H. P. per horse-power of average stream capacity.

10. Minimum and Maximum Stream Flow.

The minimum, maximum and variations in the flow stream through successive periods of time, either months or years, are necessary information for the water power engineer, and with such information, he will be expected to calculate the size of storage reservoirs and the overflow capacity of spillway dams, with their effect on the flooding of lands.

Many formulas have been suggested by engineers for determining flood flows, some involving only the rainfall and area, while others consider the slope and shape of watershed.

Such formulas are applicable only when used under conditions similar to those on which they are based.

Fanning recommends the following formula as applicable to average New England and Middle States basins:

$$Q = 200 \frac{M^{5/6}}{M}$$

in which Q = discharge in cubic ft. per second per square mile;
M = area in square miles.

Experience shows that in this formula, the value of Q appears to increase somewhat too slowly with decrease in area. M. Cooley has proposed this other formula derived from measurements of streams of flat slopes in the upper Mississippi Valley

$$Q = 180 \frac{M^{2/3}}{M}$$

and represents those floods occurring with comparative frequency, as once in 6 or 10 years.

Professor Kuichling has developed the following formula and recommends it as applicable to the South Atlantic states:

$$Q = \frac{41.6\,(620 + M)}{24 + M}$$

Another relation is due to Col. Ryves and has been deduced from experience in India

$$Q = c\,M^{\frac{2}{3}}$$

where c is a coefficient which varies with rainfall and slope of the region under consideration. For regions where the maximum rainfall is 4 to 5 inches in 24 hours

$$c = 400 \text{ to } 500 \text{ in flat country}$$
$$= 500 \text{ to } 650 \text{ in hilly country.}$$

Still another formula proposed by M. M. Davis and Wilson[1] is

$$Q = R c M^{\frac{2}{3}}$$

where R = maximum rainfall in inches in 24 hours,
c = 100 for undulating areas,
= 200 „ mountainous areas.

Possenti has developed a formula for computing floods which takes into account the catchment area, the maximum rain in 24 hours and the topographical nature of the catchment area as well as the length of the river under consideration.

Calling c = coefficient varying from 800 to 1000,
r = the maximum rainfall in 24 hours, in meters,
l = length of river in kilometers,
m = area of accidented region in km²,
p = area of flat region of catchment area,
Q = discharge in cubic meters per second

The formula is: $$Q = \frac{cr}{l}\left(m + \frac{p}{3}\right).$$

and may be used as a check on flood calculations.

11. Graphical Representation of River Discharge.

In order to study the regimen of a stream, daily measurements must be taken either by the weir method or by means of velocity measurements.

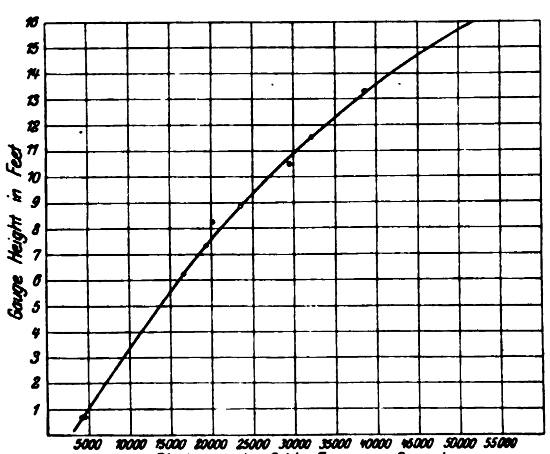


Fig. 4. Rating Curve for Wisconsin River at Kilbourn, Wis.

For a given station, where a weir or an ordinary gauge is established, if depths of water are plotted as ordinates, and for several depths of water, volume of flow is computed and plotted as abscissae, a graphical representation, also called rating curve, of the river discharge is obtained.

Fig. 4 shows a rating curve established for the Wisconsin River at Kilbourn, Wis. The small circles show the flow relative to gauge height at the time the observations were made. They were carefully taken in a fairly satisfactory section and fall on a smooth curve drawn from this data to represent the relation of gauge height to flow at similar or intervening heights.

12. The Study of Stream Flows from their Hydrographs.

The variation of stream flows are best studied from their hydrographs. These are diagrams indicating the flow rates prevailing during every day of the water year, and

[1] Irrigation Engineering — Davis and Wilson.

are easily prepared from the gauge readings. The ordinates represent the daily flow in cubic feet per second at the point of observation and the abscissæ are the time elements. Hydrographs are particularly interesting and indispensable to determine correctly the water storage for the supply of water during the dry season, and are useful also for comparison with observations made at other points on the same river.

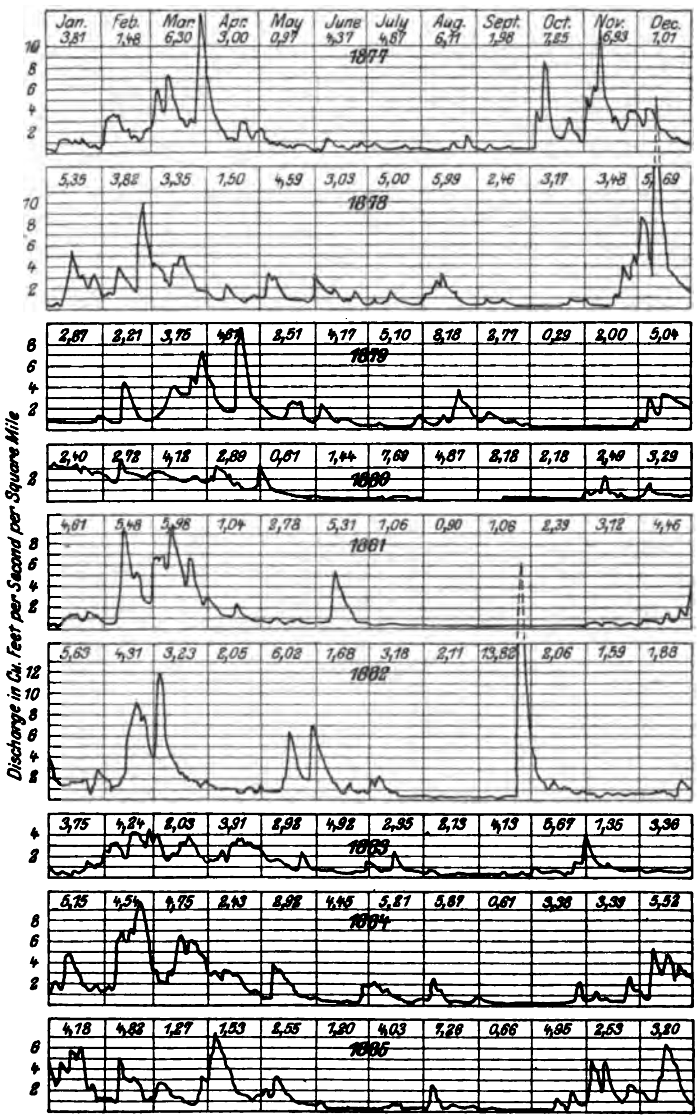


Fig. 5. Daily Flow of Passaic River, Little Falls, N. J.
Figures near top of each diagram show total monthly rainfall.

It is readily seen that if the ordinates are changed after having taken into account the fall of the river, the diagram may become a power curve. To be of any efficient value, hydrographs must be plotted for over a long period of time.

Fig. 5 and 6 show hydrographs of the Passaic River for seventeen years. The actual variations in flow as they have occured on the drainage area during that time are noted thereon. The total monthly rainfall on the drainage area has also been shown on the diagrams.

13. Influence of Forests on Rainfall and Run-off.

Many differences of opinion have been expressed as to the probable influence of forests upon rainfall, but Dr. Hough observed long ago the occurrence of "unseasonable and prolonged droughts with other vicissitudes of

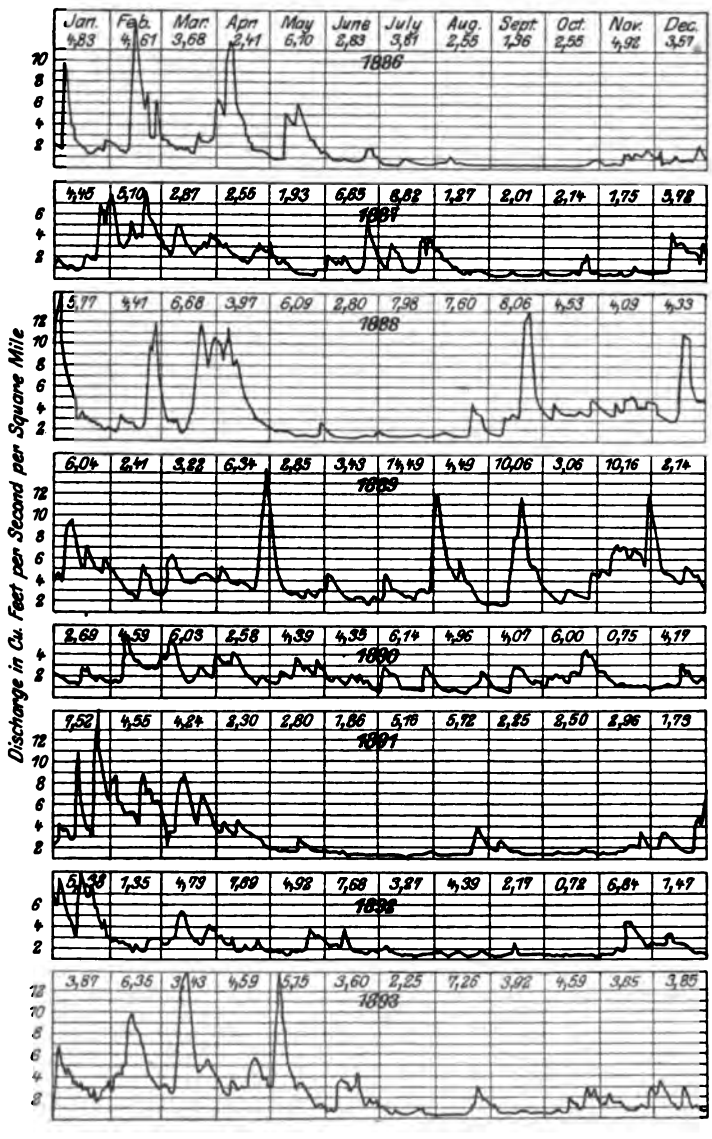


Fig. 6. Daily Flow of Passaic River, Little Falls, N. J.
Figures near top of each diagram show total monthly rainfall.

climate which it is alleged did not occur when the country was covered with forests".

As regards the effect of forests upon run-off, it is idle to say, in face of the French floods of a few years ago, that the facts tend to show that their influence is unquestionable, being directly concerned with ground and surface.

Messrs. Hoyt and Grover state that "the ground storage is increased on account of the greater receptivity of a soil loosened and opened by roots, and a surface covered wih fallen leaves and litter. The roots and cover retard the flow of water over the surface and thereby promote the absorption of the water by the soils. The effects of forests on ground storage is therefore a minimum on open sandy soils, which would readily absorb water under all conditions, and a maximum on heavy compact soils and clays, which do not take up water easily".

"The difference between evergreen and deciduous forests or between large and small trees in their effects on ground storage are probably small. Young trees and bushes, or even grass and weeds, produce approximately the same effects in this respect as virgin forests. In their effect on temperature and resultant evaporation, and especially on snow storage, the differences in forests are great. In the latter respect especially, dense evergreen growths, have much more marked effects than open or deciduous growths. Unfortunately the magnitude of their effects have not been measured, and the conclusions in regard to them are based rather on general observations than on measured differences in stream flow".

There can be no doubt that forests do act as large natural reservoirs, and to a certain extent, their action is as advantageous as that of such reservoirs in preventing floods. In the absence of forests, the run-off is very rapid, the upper layer of the soil becomes quickly saturated and the result is obvious. Deforestation therefore means lessening of protection against rapid run-off. It should be noted however, that the action of a forest as a storage reservoir is detrimental at certain times of the year. The forestry proposition has nevertheless been the subject of interminable discussion. Its nature is complex and its influence extends to other factors, as temperature and evaporation. The subject is of far-reaching importance; its proper handling depends on the relative values of all elements concerned in each particular case. It must not be concluded hastily that the area of forest growth must be directly proportional to the amount of fluctuation in comparing stream flows, but all circumstances should be known, there would be then no liability to attribute to influence of forests what really is caused by topography and surface cover of the drainage areas.

14. Influence of Lakes.

If lakes are located within the catchment area of a stream, the run-off of the latter is readily affected by them. For instance, flood flows are likely to be much smaller than they otherwise would be, for such flows are always retained temporarily by pondage in the lakes.

To illustrate how natural reservoirs tend to prevent floods, M. Rafter mentions that the configuration of the Cayuga outlet (Oswego River basin) with relation to the Clyde River, is such that frequently when there are heavy rainfalls in the catchment area of said river, its entire flood flow is discharged into Cayuga Lake, without affecting Seneca River below the mouth of Clyde River at all.

15. Evaporation as a Factor affecting Storage Reservoirs.

In arid regions, the stream flow being generally erratic, it becomes necessary to store the water for use during the months of July, August and September, inasmuch as during this period, rainfall is comparatively so little that the loss due to evaporation cannot be replaced by it.

Table V.

Name of Impounding Dam.	Area of Reservoir. Acres.	Capacity of Reservoir. Acre-Feet.	Annual Evaporation. Inches.	Annual Evaporation. °/₀ Cap.	Altitude. Feet.
Assiout Dam. Egypt. . . .	—	925,000	—	21.5	—
Laramie Natural Res. Wyo.	13,651	414,000	—	34.3	—
Rio Grande, Texas	7,965	253,368	7 ft.[1]	—	4325
Buena Vista Lake, Cal. . .	25,000	170,000	—	70.0[1]	260
San Mateo, Cal.	—	89,000	20[1] to 35	—	170
Tansa Dam, India.	5,120	62,670	—	25.2	—
Bear Valley, Cal.	1,691	40,000	27[1]	20[2] to 14	6200
Sweetwater Dam, Cal. . . .	895	22,566	54	15.0	140
Cuyamaca Dam, Cal. . . .	959	11,410	56,7	25.5	4850
Seligman, Ariz.	25.5	703	10,95 Ft.	6.8	5384
Ash Fork, Cal.	—	113.6	—	40[1] to 50	5445

[1] Estimated.

[2] 20°/₀ if no water were drawn out; 14°/₀ if there is a uniform draft of 2500 acre feet per month, from March to November inclusive.

Table VIa. *Evaporation observed by the U. S. Weather Bureau.*
(Measurements in inches)

Station	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Year
Salton Sea, 1500 Ft. inland	5.1	7.4	12.5	15.7	19.0	21.5	22.2	18.5	15.5	13.2	7.5	6.4	164.5
Salton Sea, 500 Ft. at sea . . .	3.6	5.0	6.8	9.0	11.0	13.5	14.8	12.5	12.4	9.2	6.2	4.7	108.6
Indio, Cal. . .	3.2	5.1	7.5	12.0	15.8	16.1	16.3	13.8	12.4	8.9	5.2	3.0	119.3
Mecca, Cal. . .	2.9	5.0	8.1	10.9	12.7	14.2	15.2	13.2	10.3	8.2	4.1	3.0	107.8
Brawley, Cal. .	3.1	5.0	8.0	10.7	13.8	13.7	14.1	11.3	10.2	7.0	4.1	2.7	103.6
Mammoth Cal. .	4.2	5.7	9.0	12.0	15.5	16.8	18.0	13.7	12.2	9.5	5.3	3.7	125.5
Granite Reef, Ariz	4.2	4.4	5.2	7.0	9.5	12.0	12.8	12.5	11.0	8.3	6.6	4.2	97.7
Yakima, Wash.	1.8	2.5	6.2	7.9	8.4	8.9	10.7	9.4	5.5	3.2	2.0	1.5	68.0
Hermiston, Oreg.	1.2	1.2	3.0	7.3	7.9	9.5	12.0	11.1	7.4	3.9	2.0	1.5	68.0
Minidoka, Idaho	2.2	2.5	4.0	7.0	11.2	12.3	15.0	13.5	11.0	8.5	5.8	3.5	96.5
Deer Flat, Idaho	2.0	2.8	4.2	6.0	7.9	9.6	10.6	12.2	9.2	5.4	5.5	2.0	77.4
Dutch Flat, Neb.	1.8	1.8	3.0	4.5	6.2	8.0	11.0	9.4	7.4	5.6	4.0	3.0	65.7
Kachess, Wash.	0.5	0.5	1.2	2.6	3.8	5.5	5.9	5.5	4.4	1.5	0.8	0.5	32.8
Klamath, Ore. .	0.5	1.2	3.6	6.6	7.2	7.0	8.0	9.2	6.1	2.5	1.0	0.5	53.4
Fallon, Nev. . .	1.8	1.8	2.2	3.2	5.2	7.9	9.9	8.7	5.1	3.4	2.5	2.0	53.6
Tahoe, Cal. . .	1.8	1.8	1.8	2.0	3.0	4.2	6.2	7.1	6.2	3.6	2.6	2.0	42.2
Elephant Butte, N. M.	2.5	2.7	4.5	8.0	11.5	13.4	11.6	10.5	8.6	6.8	3.9	3.0	87.0
Carlsbad N. M. .	5.0	5.2	9.0	11.1	11.0	9.1	10.6	9.3	7.8	5.9	5.4	5.0	94.4
Avalon N. M. .	4.5	4.5	5.5	7.5	10.1	11.0	12.9	12.0	9.5	7.0	5.8	4.5	94.8
Birmingham, Ala.	1.5	1.5	2.2	4.4	5.9	7.3	7.4	7.3	6.0	4.0	2.3	1.5	51.3
California, Ohio	1.0	1.5	2.5	4.1	5.1	6.2	7.2	7.3	5.6	3.0	1.5	1.0	46.0
Lawrence, Kan.	—	—	6.0	5.7	5.7	8.4	11.1	7.0	5.4	4.1	3.1	—	—

It is sometimes necessary to take in consideration the amount of water lost to the air, in the project of artificial reservoirs. Experience shows that the relative amount of evaporation is smaller if the reservoirs are of great average depth.

Table V indicates that the percentage of evaporation differs widely, but it is probable that the variations are due to difference in depth, exposure and such other factors as affect evaporation and which have been indicated before.

Table VIa shows the evaporation in inches per month and year in different parts of the United States, as observed by the U. S. Weather Bureau.

16. Units of Measurement and Conversion Tables[1].

The volume flowing in a stream is usually expressed in cubic feet per second, briefly expressed as "second-feet". A second-foot may be defined as the body of water flowing in a stream 1 foot wide, 1 foot deep, at a speed, of 1 foot per second.

The unit of capacity used in connection with storage is the acre foot and is equal to 43,560 cubic feet. It is a quantity that would cover an acre to a depth of one foot.

There is a convenient relation between the second foot and the acre foot; one second foot flowing for 24 hours will deliver 86,400 cubic feet, which equals 1.9835 acre feet, or approximately 2 acre feet.

Table VI. *Second-feet per day into acre-feet.*

1 second-foot flow for one day $= 86{,}400$ cubic feet $= \frac{86{,}400}{43{,}560}$, or 1.983471 acre-feet.

[In acre-feet.]

1 second-foot for 24 hours	1.98347	6 second-feet for 24 hours	11.90083
2 second-feet for 24 hours	3.96694	7 second-feet for 24 hours	13.88430
3 second-feet for 24 hours	5.95041	8 second-feet for 24 hours	15.86777
4 second-feet for 24 hours	7.93388	9 second-feet for 24 hours	17.85124
5 second-feet for 24 hours	9.91735		

	0.	1.	2.	3.	4.	5.	6.	7.	8.	9.
0	—	1.983	3.967	5.950	9.934	9.917	11.901	13.884	15.868	17.851
1	19.835	21.818	23.802	25.785	27.769	29.752	31.736	33.719	35.702	37.686
2	39.669	41.653	43.636	45.620	47.603	49.587	51.570	53.554	55.537	57.521
3	59.504	61.488	63.471	65.455	67.438	69.421	71.405	73.388	75.372	77.355
4	79.339	81.322	83.306	85.289	87.273	89.256	91.240	93.223	95.207	97.190
5	99.174	101.157	103.140	105.124	107.107	109.091	111.074	113.058	115.041	117.025
6	119.008	120.992	122.975	124.959	126.942	128.926	130.909	132.892	134.876	136.859
7	138.843	140.826	142.810	144.793	146.777	148.760	150.744	152.728	154.711	156.694
8	158.678	160.661	162.645	164.628	166.611	168.595	170.578	172.562	174.545	176.529
9	178.512	180.496	182.479	184.463	186.446	188.430	190.413	192.397	194.380	196.364

As the months are of varying length it is necessary to use three or four factors to convert the average discharge for the month in second feet into the total in acre feet. The number of days in the month will give the total monthly discharge in acre feet. The daily quantity therefore must be

[1] Hydrographic Manual. U. S. Geological Survey.

multiplied by 28 for the month of February, or 29 for that month in leap year, and by 30 or 31 for the other months.

For the month of February when it has 28 days, the factor to be used is 55.537188. For convenience in computation this factor multiplied from 1 to 9 is given as follows:

1	55.53719	4	222.14872	7	388.76026
2	111.07436	5	277.68590	8	444.29744
3	166.61154	6	333.22308	9	499.83462

When February has 29 days the factor to be used is 57.520659. This multiplied from 1 to 9 is as follows:

1	57.52066	4	230.08264	7	402.64462
2	115.04132	5	287.60330	8	460.16528
3	172.56198	6	345.12396	9	517.68594

For the months containing 30 days, viz, April, June, September and November, the factor to be used is 59.50413.

This when multiplied by the unit figures is as follows:

1	59.50413	4	238.01652	7	416.52891
2	119.00826	5	297.52065	8	476.03304
3	178.51239	6	357.02478	9	535.53717

For the months containing 31 days, viz, January, March, May, July, August, October and December, the factor to be used is 61.487601.

This when multiplied by the unit figures is as follows:

1	61.48760	4	245.95040	7	430.41321
2	122.97520	5	307.43800	8	491.90081
3	184.46280	6	368.92561	9	553.38841

The run-off per square mile is obtained by simply dividing the average for the month by the total number of square miles in the drainage basin, which is usually ascertained by planimeter measurements from the best map available. Being a rate of flow it is independent of time, and therefore the number of days in each month does not enter into the ratio.

The depth of run-off over the drainage basin is usually computed in inches for convenience of comparison with the depth of rainfall which is almost invariably given in that unit. This depth can most conveniently be computed from the run-off per square mile by computation based on the number of days in each month and the relation between the rate of flow and the depth in inches for this quantity were it held during the given number of days. One second foot for 24 hours is equivalent to 86,400 cubic feet in one day. In other words, 1 cubic foot per second run off from 1 square mile would, if held upon this area, cover it to a depth represented by dividing 86,400 by the number of square feet in a mile, 27 878 400 or 5280 squared. Completing this division, it is found that 1 second foot for one day is equivalent to a body of water covering 1 square mile 0,003099173 feet, or 0.037190076 inch deep. Multiplying this by the number of days in a month gives the following factors:

28 days	1.041322528
29 days	1.078512604
30 days	1.115702680
31 days	1.152892756

Table VII. *Second-feet into acre-feet per month.*

Second-feet.	Acre-feet per 30 days.	Acre-feet per 31 days.	Second-feet.	Acre-feet per 30 days.	Acre-feet per 31 days.
1	59.504	61.488	51	3,034.711	3,135.868
2	119.008	122.975	52	3,094.215	3,197.355
3	178.512	184.463	53	3,153.719	3,258.843
4	238.017	245.950	54	3,213.223	3,320.330
5	297.521	307.438	55	3,272.727	3,381.818
6	357.025	368.926	56	3,332.231	3,443.306
7	416.529	430.413	57	3,391.735	3,504.793
8	476.033	491.901	58	3,451.240	3,566.281
9	535.537	553.388	59	3,510.744	3,627.768
10	595.041	614.876	60	3,570.248	3,689.256
11	654.545	676.364	61	3,629.752	3,750.744
12	714.050	737.851	62	3,689.256	3,812.231
13	773.554	799.339	63	3,748.760	3,873.719
14	833.058	860.826	64	3,808.264	3,935.206
15	892.562	922.314	65	3,867.768	3,996.694
16	952.066	983.802	66	3,927.273	4,058.182
17	1,011.570	1,045.289	67	3,986.777	4,119.669
18	1,071.074	1,106.777	68	4,046.281	4,181.157
19	1,130.578	1,168.264	69	4,105.785	4,242.644
20	1,190.083	1,229.752	70	4,165.289	4,304.132
21	1,249.587	1,291.240	71	4,224.793	4,365.620
22	1,309.091	1,352.727	72	4,284.297	4,427.107
23	1,368.595	1,414.215	73	4,343.801	4,488.595
24	1,428.099	1,475.702	74	4,403.306	4,550.082
25	1,487.603	1,537.190	75	4,462.810	4,611.570
26	1,547.107	1,598.678	76	4,522.314	4.673.058
27	1,606.612	1,660.165	77	4,581.818	4,734.545
28	1,666.116	1,721.653	78	4,641.322	4,796.033
29	1,725.620	1,783.140	79	4,700.826	4,857.520
30	1,785.124	1,844.628	80	4,760.330	4,919.008
31	1,844.628	1,906.116	81	4,819.835	4,980.496
32	1,904.132	1,967.603	82	4,879.339	5,041.983
33	1,963.636	2,029.091	83	4,938.843	5,103.471
34	2,023.140	2,090.578	84	4,998.347	5,164.958
35	2,082.645	2,152.066	85	5,057.851	5,226.446
36	2,142.149	2,213.554	86	5,117.355	5,287.934
37	2,201.653	2,275.041	87	5,176.859	5,349.421
38	2,261.157	2,236.529	88	5,236.363	5,410.909
39	2,320.661	2,398.016	89	5,295.868	5,472.396
40	2,380.165	2,459.504	90	5,355.372	5,533.884
41	2,439.669	2,520.992	91	5,414.876	5,595.372
42	2,499.173	2,582.479	92	5.474.380	5,656.859
43	2,558.678	2,643.967	93	5,533.884	5,718.347
44	2,618.182	2,705.454	94	5,593.388	5,779.834
45	2,677.686	2,766.942	95	5,652.892	5,841.322
46	2,737.190	2,828.430	96	5,712.396	5,902.810
47	2,796.694	2,889.917	97	5,771.901	5,964.297
48	2,856.198	2,951.405	98	5,831.405	6,025.785
49	2,915.702	3,012.892	99	5,890.909	6,087.272
50	2,975.207	3,074.380	100	5,950.413	6,148.760

Table VIII. *Run-off in second-feet per square mile into depth in inches per month of 30 and 31 days.*

Second-feet per square mile.	30 days.	31 days.	Second-feet per square mile.	30 days.	31 days.	Second-feet per square mile.	30 days.	31 days.	Second-feet per square mile.	30 days.	31 days.
	Inch.	*Inch.*		*Inch.*	*Inch.*		*Inch.*	*Inch.*		*Inch.*	*Inch.*
0.01	0.01	0.01	.61	.68	.70	1.21	1.35	1.39	1.81	2.02	2.09
.02	.02	.02	.62	.69	.71	1.22	1.36	1.41	1.82	2.03	2.10
.03	.03	.03	.63	.70	.72	1.23	1.37	1.42	1.83	2.04	2.11
.04	.04	.04	.64	.71	.74	1.24	1.38	1.43	1.84	2.05	2.12
.05	.06	.06	.65	.72	.75	1.25	1.39	1.44	1.85	2.06	2.13
.06	.07	.07	.66	.73	.76	1.26	1.41	1.45	1.86	2.08	2.14
.07	.08	.08	.67	.74	.77	1.27	1.42	1.46	1.87	2.09	2.16
.08	.09	.09	.68	.75	.78	1.28	1.43	1.48	1.88	2.10	2.17
.09	.10	.10	.69	.77	.79	1.29	1.44	1.49	1.89	2.11	2.18
.10	.11	.12	.70	.78	.81	1.30	1.45	1.50	1.90	2.12	2.19
.11	.12	.13	.71	.79	.82	1.31	1.46	1.51	1.91	2.13	2.20
.12	.13	.14	.72	.80	.83	1.32	1.47	1.52	1.92	2.14	2.21
.13	.14	.15	.73	.81	.84	1.33	1.48	1.53	1.93	2.15	2.22
.14	.16	.16	.74	.82	.85	1.34	1.50	1.54	1.94	2.16	2.24
.15	.17	.17	.75	.83	.86	1.35	1.51	1.56	1.95	2.18	2.25
.16	.18	.18	.76	.84	.87	1.36	1.52	1.57	1.96	2.19	2.26
.17	.19	.20	.77	.85	.89	1.37	1.53	1.58	1.97	2.20	2.27
.18	.20	.21	.78	.87	.90	1.38	1.54	1.59	1.98	2.21	2.28
.19	.21	.22	.79	.88	.91	1.39	1.55	1.60	1.99	2.22	2.29
.20	.22	.23	.80	.89	.92	1.40	1.56	1.61	2.00	2.23	2.31
.21	.23	.24	.81	.90	.93	1.41	1.57	1.63	2.01	2.24	2.32
.22	.24	.25	.82	.91	.94	1.42	1.58	1.64	2.02	2.25	2.33
.23	.26	.26	.83	.92	.95	1.43	1.60	1.65	2.03	2.26	2.34
.24	.27	.28	.84	.93	.97	1.44	1.61	1.66	2.04	2.28	2.35
.25	.28	.29	.85	.94	.98	1.45	1.62	1.67	2.05	2.29	2.36
.26	.29	.30	.86	.95	.99	1.46	1.63	1.68	2.06	2.30	2.37
.27	.30	.31	.87	.97	1.00	1.47	1.64	1.69	2.07	2.31	2.39
.28	.31	.32	.88	.98	1.01	1.48	1.65	1.71	2.08	2.32	2.40
.29	.32	.33	.89	.99	1.02	1.49	1.66	1.72	2.09	2.33	2.41
.30	.33	.35	.90	1.00	1.04	1.50	1.67	1.73	2.10	2.34	2.42
.31	.35	.36	.91	1.01	1.05	1.51	1.68	1.74	2.11	2.35	2.43
.32	.36	.37	.92	1.02	1.06	1.52	1.70	1.75	2.12	2.37	2.44
.33	.37	.38	.93	1.03	1.07	1.53	1.71	1.76	2.13	2.38	2.46
.34	.38	.39	.94	1.04	1.08	1.54	1.72	1.78	2.24	2.39	2.47
.35	.39	.40	.95	1.05	1.09	1.55	1.73	1.79	2.15	2.40	2.48
.36	.40	.41	.96	1.07	1.10	1.56	1.74	1.80	2.16	2.41	2.49
.37	.41	.43	.97	1.08	1.12	1.57	1.75	1.81	2.17	2.42	2.50
.38	.43	.44	.98	1.09	1.13	1.58	1.76	1.82	2.18	2.43	2.51
.39	.44	.45	.99	1.10	1.14	1.59	1.77	1.83	2.19	2.44	2.52
.40	.45	.46	1.00	1.11	1.15	1.60	1.79	1.84	2.20	2.45	2.54
.41	.46	.47	1.01	1.13	1.16	1.61	1.80	1.86	2.21	2.47	2.55
.42	.47	.48	1.02	1.14	1.18	1.62	1.81	1.87	2.22	2.48	2.56
.43	.48	.49	1.03	1.15	1.19	1.63	1.82	1.88	2.23	2.49	2.57
.44	.49	.51	1.04	1.16	1.20	1.64	1.83	1.89	2.24	2.50	2.58
.45	.50	.52	1.05	1.17	1.21	1.65	1.84	1.90	2.25	2.51	2.59
.46	.52	.53	1.06	1.18	1.22	1.66	1.85	1.91	2.26	2.52	2.61
.47	.53	.54	1.07	1.19	1.23	1.67	1.86	1.93	2.27	2.53	2.62
.48	.54	.55	1.08	1.20	1.25	1.68	1.87	1.94	2.28	2.54	2.63
.49	.55	.56	1.09	1.22	1.26	1.69	1.89	1.95	2.29	2.55	2.64
.50	.56	.58	1.10	1.23	1.27	1.70	1.90	1.96	2.30	2.57	2.65
.51	.57	.59	1.11	1.24	1.28	1.71	1.91	1.97	2.31	2.58	2.66
.52	.58	.60	1.12	1.25	1.29	1.72	1.92	1.98	2.32	2.59	2.67
.53	.59	.61	1.13	1.26	1.30	1.73	1.93	1.99	2.33	2.60	2.69
.54	.60	.62	1.14	1.27	1.31	1.74	1.94	2.01	2.34	2.61	2.70
.55	.61	.63	1.15	1.28	1.33	1.75	1.95	2.02	2.35	2.62	2.71
.56	.62	.64	1.16	1.29	1.34	1.76	1.96	2.03	2.36	2.63	2.72
.57	.63	.66	1.17	1.31	1.35	1.77	1.97	2.04	2.37	2.64	2.73
.58	.64	.67	1.18	1.32	1.36	1.78	1.99	2.05	2.38	2.66	2.74
.59	.65	.68	1.19	1.33	1.37	1.79	2.00	2.06	2.39	2.67	2.76
.60	.67	.69	1.20	1.34	1.38	1.80	2.01	2.08	2.40	2.68	2.77

Table VIII (Continued). *Run-off in second-feet per square mile, etc.*

Second-feet per square mile.	30 days.	31 days.	Second-feet per square mile.	30 days.	31 days.	Second-feet per square mile.	30 days.	31 days.	Second-feet per square mile.	30 days.	31 days.
	Inch.	*Inch.*		*Inch.*	*Inch.*		*Inch.*	*Inch.*		*Inch.*	*Inch.*
2.41	2.69	2.78	2.81	3.14	3.24	3.21	3.58	3.69	3.61	4.03	4.16
2.42	2.70	2.79	2.82	3.15	3.25	3.22	3.59	3.71	3.62	4.04	4.17
2.43	2.71	2.80	2.83	3.16	3.26	3.23	3.60	3.72	3.63	4.05	4.18
2.44	2.72	2.81	2.84	3.17	3.27	3.24	3.61	3.74	3.64	4.06	4.20
2.45	2.73	2.82	2.85	3.18	3.29	3.25	3.63	3.75	3.65	4.07	4.21
2.46	2.74	2.84	2.86	3.19	3.30	3.26	3.64	3.76	3.66	4.08	4.22
2.47	2.76	2.85	2.87	3.20	3.31	3.27	3.65	3.77	3.67	4.09	4.23
2.48	2.77	2.86	2.88	3.21	3.32	3.28	3.66	3.78	3.68	4.11	4.24
2.49	2.78	2.87	2.89	3.22	3.33	3.29	3.67	3.79	3.69	4.12	4.25
2.50	2.79	2.88	2.90	3.24	3.34	3.30	3.68	3.80	3.70	4.13	4.27
2.51	2.80	2.89	2.91	3.25	3.35	3.31	3.69	3.82	3.71	4.14	4.28
2.52	2.81	2.91	2.92	3.26	3.37	3.32	3.70	3.83	3.72	4.15	4.29
2.53	2.82	2.92	2.93	3.27	3.38	3.33	3.71	3.84	3.73	4.16	4.30
2.54	2.83	2.93	2.94	3.28	3.39	3.34	3.73	3.85	3.74	4.17	4.31
2.55	2.85	2.94	2.95	3.29	3.40	3.35	3.74	3.86	3.75	4.18	4.32
2.56	2.86	2.95	2.96	3.30	3.41	3.36	3.75	3.87	3.76	4.20	4.33
2.57	2.87	2.96	2.97	3.31	3.42	3.37	3.76	3.89	3.77	4.21	4.35
2.58	2.88	2.97	2.98	3.32	3.44	3.38	3.77	3.90	3.78	4.22	4.36
2.59	2.89	2.99	2.99	3.34	3.45	3.39	3.78	3.91	3.79	4.23	4.37
2.60	2.90	3.00	3.00	3.35	3.46	3.40	3.79	3.92	3.80	4.24	4.38
2.61	2.91	3.01	3.01	3.36	3.47	3.41	3.80	3.93	3.81	4.25	4.39
2.62	2.92	3.02	3.02	3.37	3.48	3.42	3.82	3.94	3.82	4.26	4.40
2.63	2.93	3.03	3.03	3.38	3.49	3.43	3.83	3.95	3.83	4.27	4.42
2.64	2.95	3.04	3.04	3.39	3.50	3.44	3.84	3.97	3.84	4.28	4.43
2.65	2.96	3.06	3.05	3.40	3.52	3.45	3.85	3.98	3.85	4.30	4.44
2.66	2.97	3.07	3.06	3.41	3.53	3.46	3.86	3.99	3.86	4.31	4.45
2.67	2.98	3.08	3.07	3.43	3.54	3.47	3.87	4.00	3.87	4.32	4.46
2.68	2.99	3.09	3.08	3.44	3.55	3.48	3.88	4.01	3.88	4.33	4.47
2.69	3.00	3.10	3.09	3.45	3.56	3.49	3.89	4.02	3.89	4.34	4.48
2.70	3.01	3.11	3.10	3.46	3.57	3.50	3.90	4.04	3.90	4.35	4.50
2.71	3.02	3.12	3.11	3.47	3.59	3.51	3.92	4.05	3.91	4.36	4.51
2.72	3.03	3.14	3.12	3.48	3.60	3.52	3.93	4.06	3.92	4.37	4.52
2.73	3.05	3.15	3.13	3.49	3.61	3.53	3.94	4.07	3.93	4.38	4.53
2.74	3.06	3.16	3.14	3.50	3.62	3.54	3.95	4.08	3.94	4.40	4.54
2.75	3.07	3.17	3.15	3.51	3.63	3.55	3.96	4.09	3.95	4.41	4.55
2.76	3.08	3.18	3.16	3.53	3.64	3.56	3.97	4.10	3.96	4.42	4.57
2.77	3.09	3.19	3.17	3.54	3.65	3.57	3.98	4.12	3.97	4.43	4.58
2.78	3.10	3.21	3.18	3.55	3.67	3.58	3.99	4.13	3.98	4.44	4.59
2.79	3.11	3.22	3.19	3.56	3.60	3.59	4.01	4.14	3.99	4.45	4.60
2.80	3.12	3.23	3.20	3.57	3.68	3.60	4.02	4.15	4.00	4.46	4.61

Table IX. *Second-feet per square mile into depth in inches per month.*

Second-feet per square mile.	28 days.	29 days.	30 days.	31 days.
	Inches.	*Inches.*	*Inches.*	*Inches.*
1	1.041	1.079	1.116	1.153
2	2.083	2.157	2.231	2.306
3	3.124	3.236	3.347	3.459
4	4.165	4.314	4.463	4.612
5	5.207	5.393	5.579	5.764
6	6.248	6.471	6.694	6.917
7	7.289	7.550	7.810	8.070
8	8.331	8.628	8.926	9.223
9	9.372	9.707	10.041	10.376
10	10.413	10.785	11.157	11.529

Chapter II. Stream Measurements.

17. Stream Measurements.

The measurement of river discharge is a work of capital importance, it consists in the computation of the exact power factor of a river. As a matter of fact, in order to estimate the power of a stream, it is necessary to know its discharge and its slope. The latter is determined without difficulty, a simple survey answering the question. But the computation of the discharge is a very different matter. This does not imply that the study offers difficulties, but as the flow of a river is a very variable element, its determination necessitates very delicate observations and precise operations, and these can only be accomplished by having had considerable previous experience, and by exercising extraordinary care. Not only the extreme low, mean, and high water discharges are required, but also their respective relative time of duration must be determined; therefore such work should be carried on for several years; it is then possible to fix some fair average, and also consider the years of minimum flow for the establishment of auxiliary steam power, if this be unavoidable.

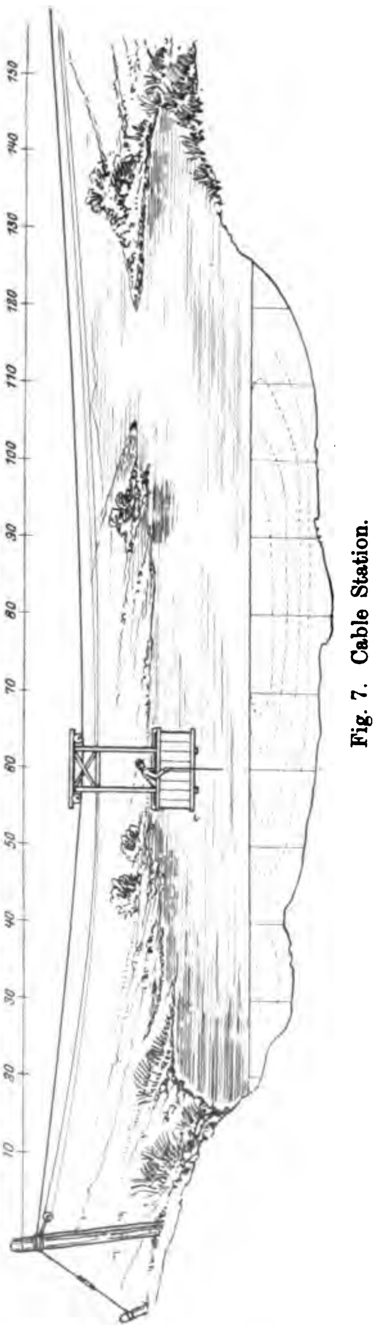


Fig. 7. Cable Station.

18. Area of River Cross-section.

In order to measure the discharge of a stream it is necessary to determine the area of cross-section and compute the velocity of the flowing water. A profile of the river bed, taken in a perpendicular to the direction of the flow, is best attained by making soundings at equal intervals, and using a tagged rope stretched across the channel on the very site to be plotted. For depths of 4 feet, it is simpler to wade into the water, and use a rod graduated to a tenth of a foot. In case of depths of more than 4 feet a boat may be used, for making the rod soundings; if the gaging station is to be permanent, a car, supported by a steel cable extended across the river, is used to best advantage (Fig. 7).

If the measurement of the velocity is to be made with floats, it is necessary to determine the cross-section of the river at several places equally spaced, and then the mean of the measured areas is taken. The contour of the river bed, as has been said, is an important factor for the computation of discharge, but as the surface of the water fluctuates, it is also necessary to refer this surface to a gauge, thereby making it possible to determine the discharge of the river at any stage.

The following method, very simple in its application, may be used instead of a planimeter. The curve is plotted with its respective axes on a paper of homogeneous grain, and then cut all along the perimeter, leaving absolutely free the figure area which is to be measured. This is then weighed in a balance of precision. Call W its weight and A its area.

A small area of comparison, square or rectangular in shape, the dimensions of which have been accurately determined, is then cut from the same paper, and weighed with the same balance and in the same conditions. Call A_1 its area, and W_1 its weight in milligrams. The areas A_1 and A, having been cut from a paper of uniform surface and density, their surfaces are proportional to their volumes, and consequently to their weights:

$$\frac{A}{A_1} = \frac{W}{W_1} \quad \text{or} \quad A = \frac{W}{W_1} A_1$$

The value of A is then computed to the natural scale to which the curve has been drawn.

19. Float Measurements.

This method is used principally on large rivers of more or less uniform cross-section. The site must be chosen where the flow is regular, that is to say not presenting eddy-currents or rapids. The upper and lower cross-sections of the river having been marked, and a rope stretched across the stream, the float is thrown into the water some distance upstream from the first cross-section, and the time of passage between the two marked sections is noted. The velocity is then recorded in feet per second. It is better to repeat the operation at different distances from the shore, thereby determinating the average velocity. If rods or tubes are used as floats, they must be calibrated so that they will float in a vertical position, the lower end clearing the bottom of the river, the upper end emerging above water level so as to be plainly visible to the observer. This method, considering the many factors which tend to alter its results, is used where relatively close approximations are sufficient, and although it does not afford much accuracy, the 80 per cent of the velocity as determined may be considered as the average velocity of the water in the stream.

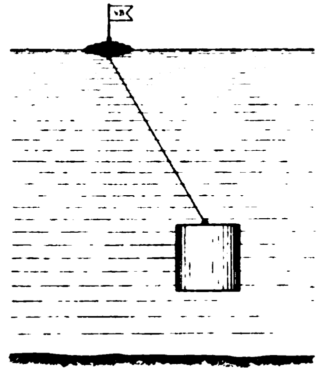


Fig. 8. Double Float used by Ellis in Connecticut River Survey in 1874.

20. Slope Measurements.

The slope of section of river is the ratio of its vertical fall to the horizontal length of that section. The slope of large streams is generally small, and it is customary to consider the inclined length and horizontal length as practically coinciding.

In determining the discharge of a river by the slope method, the engineer must rely principally on experience and good judgement. As a matter of fact, the results obtained through this sort of measurements are only approximate, but close enough for reconnaissance work. The method may be used also as a check for measurements made by the float and meter methods. In order to compute the discharge of a river in this way, the Chezy formula, with the Kutter coefficient C is used:

$$v = C\sqrt{RS} \text{ in which } C = \frac{41.66 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.66 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}$$

The use of this formula makes it a necessity to measure the slope of the water surface and to determine the cross-section of the river bed at several places, in order to obtain a mean hydraulic radius, and give a proper value to the coefficient of roughness n.

The value of n may be taken as

0.020 to 0.025 for smooth sandy and fine gravel beds:
0.030 to 0.035 for rough beds; and
0.040 to 0.055 when the banks are overflowed and the river bed covered with detritus and aquatic plants.

It is necessary to observe that in natural streams, especially if their cross-section is rapidly enlarged during high water stages by overflowing adjascent lands, the low water level's slope is steeper than the corresponding flood level inclination for a determined section. If an obstruction, a spillway dam for instance, is encountered by the water, the flood slope will be more inclined than the low water slope. Generally, except in the case of obstruction, and for estimating water power at a definite locality, the natural slope may be taken as one and the same for all stages of the river.

21. Current Meter Measurements.

The method of gaging streams by means of the current meter is that which provides most accurate results. There are several types of instruments, the Price, Haskell and Fteley (Fig. 9, 10, 11) being the most commonly used. The recording-devices of these meters are either electrical or mechanical. Experience shows that the Fteley meter is the most accurate of all, within the limits of its adaptability, although the Haskell meter is better used for measurements on large rivers. In making meter measurements, the instrument must be examined frequently, and any grass or moss lodged around the revolving shaft must be removed.

The velocity in a vertical may be determined in three ways:

1st: by measuring the velocity at a depth corresponding to the approximate position of the thread of average velocity;
2nd: by the vertical velocity-curve method;
3rd: by integration.

First: The position of the thread of mean velocity has been determined after numerous experiments, and the relative depths established as follows[1]:

C. C. Babb	Depth	0.58
Humphrey & Abbott	"	0.63
T. G. Ellis	"	0.64
Wheeler & Lynch	"	0.67

[1] U. S. Geological Survey W. S. Paper, No. 56.

In practice, the meter is held at a point located at 0.6 of the total depth. The error resulting in using 0.6 depth is very small and ranges from — 6% to + 4% with a mean of 0 per cent[1], but this value may be increased to better advantage to 0.67 depth when measuring canals and flumes or narrow and deep natural channels.

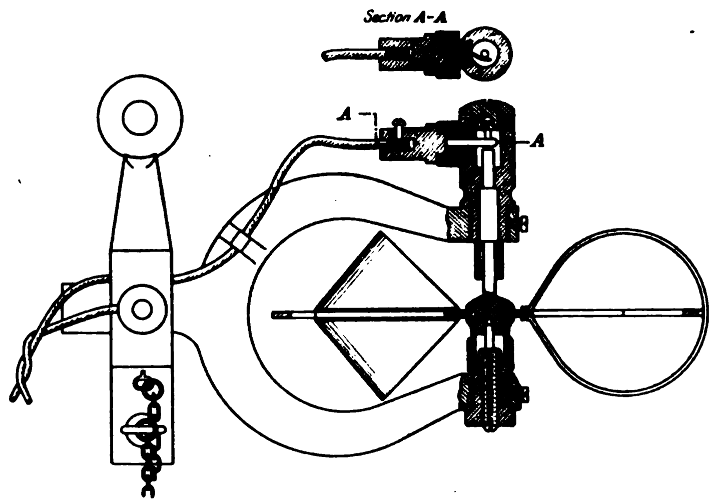


Fig. 9. Cross-Section of small Price Electric Current Meter, showing Details.

Second: The vertical velocity curve is best obtained by making several measurements at regular intervals in a vertical. The velocities so obtained are plotted on cross-section paper as abscissas, the respective depths

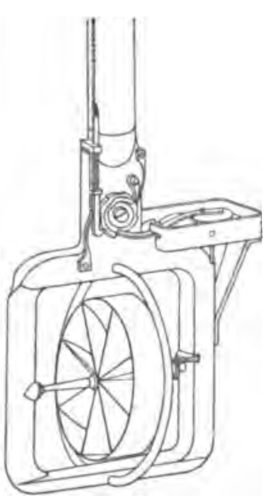


Fig. 10. Fteley Current Meter.

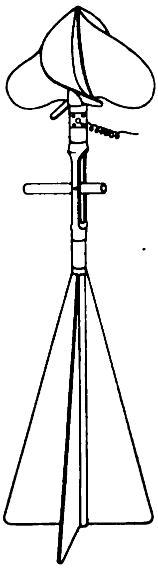


Fig. 11. Haskell Meter.

as ordinates. The mean velocity is then obtained by dividing the area bounded by this curve passing through the above plotted points and its axis by the depth. This area may be measured by a planimeter or by the weighing method.

[1] River Discharge—Hoyt & Grover.

Table X. *Vertical-velocity measurement made November 2, 1903, by E. C. Murphy, meter No. 338, on Susquehanna River, at Harrisburg, State of Pennsylvania. Measurements at 585 feet from initial point for soundings. Depth 8 ft.*

Gage height: beginning 3.08 ft., ending 3.08 ft., mean 3.08 ft. Channel open. Thickness of ice, -ft.

Field notes.				Data from curve and computation.				
Depth of center of meter below surface in feet.	Number revolutions per *50* seconds.	Number revolutions per second.	Velocity per second.	Middle of horizontal section.	Velocity from curve.	Point in vertical.	Velocity.	Coefficient for reducing to mean velocity.
0.5	*61—61*, *61—62*	*1.22*	*2.85*	1	*2.85*	0.6 depth.	*2.45*	*1.01*
1.5	*59*, *57*	*1.16*	*2.71*	2	*2.78*	1 ft. below surface.	*2.82*	*0.88*
2.5	*58*, *61*	*1.19*	*2.79*	3	*2.72*	Bottom.	*1.40*	*1.77*
3.5	*54*, *55*	*1.09*	*2.58*	4	*2.62*	Mid depth.	*2.57*	*0.96*
4.5	*57*, *54*	*1.11*	*2.60*	5	*2.50*			
5.5	*45*, *51*	*.96*	*2.25*	6	*2.35*	Depth of mean velocity=	*4.5* ft. *56* per cent of depth.	
6.5	*44*, *48*	*.92*	*2.16*	7	*2.16*			
7.5	*38*, *39*	*.77*	*1.82*	8	*1.82*			
8.0				9				
				10				
				Total	*19.80*	Computed by *Brundage.*		
				Mean	*2.48*	Checked by *Marsh.*		

Fall of river, — feet per mile.

Remarks. (Wind conditions. Character of stream bed. Roughness under surface of ice, etc.)

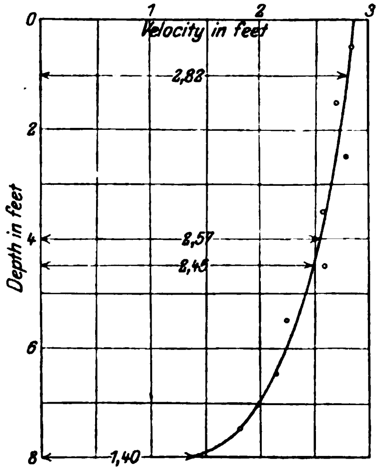


Fig. 12. Vertical-velocity Curve.

The form for recording the observations is given in table X and the velocities of the fourth column are plotted as shown on the curve, Fig. 12[1]. The mean abscissae of this curve obtained as indicated above, is the mean velocity in the vertical.

Third: The integration method consists in moving the meter from the surface to the river bed at a uniform speed, and back again to the surface, the revolutions and time being observed. The velocity is thereby mechanically integrated as the meter passes successively through all the velocities of the running stream. Care must be taken that the moving be slow, and

[1] U. S. Geological Survey Water Supply & Irrigation Paper, No. 94.

especially uniform, otherwise there are liabilities of error. However, this method is objectionable, because it offers more complications and attendance, than the other methods already described.

22. Computation of Meter Measurements.

Fig. 13 shows the cross-section of the Saline River, near Salina, Kansas. The soundings were taken at each 5 feet of width from the initial point, and the velocity was observed at 0.6 depth below the surface at each of these verticals. The computations are shortened by finding the discharge through each double strip at a time.

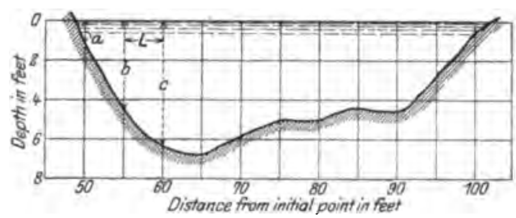


Fig. 13. Cross-section of Saline River at gauging Station near Salina, Kans.

The following formulas are used to compute mean depth, mean velocity, area, and discharge.

Let d = mean depth for a single strip,
V = mean velocity for a single strip,
d'_m = mean depth for a double strip,
V'_m = mean velocity for a double strip,
a, b, c = consecutive depths, L feet apart,
V_a, V_b, V_c = observed velocities in the verticals a, b, c,
L = the width of a single strip,
Q' = the discharge through a double strip,
Q = discharge through single strip.

Then we have:

1) $$Q = a V_a L$$

2) $$Q = \left(\frac{a + 6b + c}{8}\right) L V_b$$

3) $$V = \left(\frac{V_a + V_b}{2}\right)$$

4) $$d = \frac{a + b}{2}$$

5) $$Q = d V L = \left(\frac{a + b}{2}\right) L \left(\frac{V_a + V_b}{2}\right)$$

6) $$d'_m = \frac{a + 4b + c}{6}$$

7) $$V'_m = \frac{V_a + 4V_b + V_c}{6}$$

8) $$Q' = d'_m V'_m 2L = \left(\frac{a + 4b + c}{6}\right) 2L \left(\frac{V_a + 4V_b + V_c}{6}\right).$$

Formulae 6 and 7 are based on the assumption that the stream bed is a series of parabolic arcs, also that the horizontal velocity curves are parabolic arcs, both of which assumptions are approximately true.

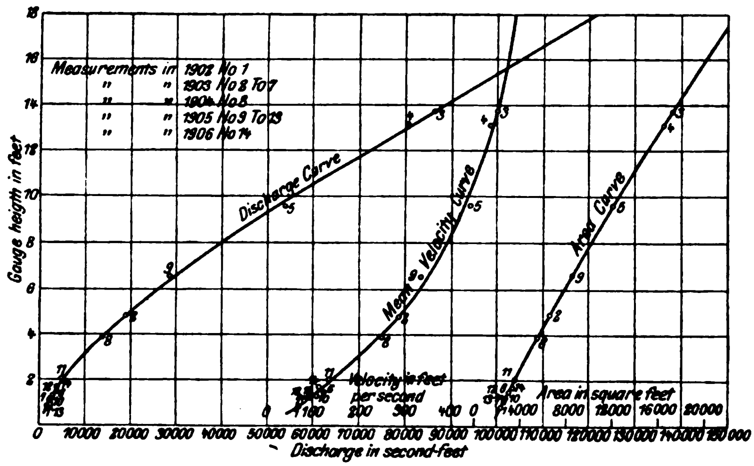


Fig. 14. Method of Plotting Curves for Discharge, Mean Velocity and Area of Rivers.

Fig. 14 shows the discharge, mean velocity and area curves for the Potomac River at Point of Rocks, Md.

The product of the area and mean velocity as shown by their relative curves, for any gauge height, equals the discharge for the same gauge height.

23. Measurements of Flow over Weirs and Dams. ***Thin edged weirs:***

In the following let

$Q =$ discharge in cubic feet per second,
$H =$ observed head where there is no velocity of approach, or head corrected for velocity of approach,
$L =$ length of weir corrected for end contractions if any,
$L' =$ actual length of crest of weir with end contractions,
$h =$ head due to velocity $= \dfrac{v^2}{2g}$
$D =$ actual depth on crest of weir, taken at a point upstream to avoid surface curve,
$P =$ height of weir crest above bottom of channel of approach.

Francis Formulae

$$Q = 3.33\, L H^{\frac{3}{2}};$$

If there are end contractions:

$$L = L' - 0.1\, N H,$$

if there is velocity of approach:

$$Q = 3.33\, L\left[(D + h)^{\frac{3}{2}} - h^{\frac{3}{2}}\right].$$

Three halves powers for numbers from 0 to 12 are given in Table XI.

Formula of Boileau

including correction for velocity of approach

$$Q = 3.3455 \frac{P + D}{\sqrt{(P + D)^2 - D^2}} L D^{\frac{3}{2}}$$

The ratio

$$\frac{P + D}{\sqrt{(P + D)^2 - D^2}}$$

is greater than 1; it decreases to 1 when D decreases to 0, and increases when D increases.

Formula of Fteley and Stearns

$$Q = 3.31 L H^{\frac{3}{2}} + 0.007 L$$

in which no correction is made for velocity of approach.

If the latter must be considered, the value of correction is determined by

$H = D + a h$ where

$a = 1.5$ for suppressed weirs,

$L = L' - 0.1 N H$ and $a = 2.05$ if there are end contractions.

Bazin Formula

$$Q = \mu \left[1 + 0.55 \left(\frac{D}{D + P}\right)^2\right] L D \sqrt{2 g D}$$

in which

$$\mu = 0.405 + \frac{0.0984}{D}.$$

The mean velocity in the channel of approach is influenced by the depth P, consequently the discharge is also influenced by the same factor.

Table XII[1] gives the discharge per foot of length over sharp crested vertical weirs, without end contractions, of heights 2, 4, 6, 8, 10, 20 and 30 feet, computed from Bazin's formula. Although this formula is based on data obtained from experiments with heads not greater than 1.64 feet, discharges for heads of 4 feet and less computed thereby, agree within 2 per cent with those obtained by the use of the Fteley and Stearns formula. The discharge given by this table is corrected for velocity of approach, and the head to be used is that observed 16 feet or more upstream from the crest of the weir.

This table is computed from the formula of Bazin as given above:

$$Q = \left(0.405 + \frac{0.0984}{D}\right) \left[1 + 0.55 \left(\frac{D}{D + P}\right)^2\right] L D \sqrt{2 g D}.$$

[1] U. S. Geological Survey Water Supply and Irrigation Paper No. 200.

Table XI. *Three-halves powers for numbers from 0 to 12.*

Hundredths \ Units	0	1	2	3	4	5	6	7	8	9	10	11
0.00	0.0000	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829
.01	.0010	1.0150	2.8497	5.2222	8.0300	11.2139	14.7337	18.5600	22.6699	27.0450	31.6702	36.5326
.02	.0028	1.0302	2.8710	5.2482	8.0601	11.2475	14.7705	18.5997	22.7123	27.0890	31.7177	36.5824
.03	.0052	1.0453	2.8923	5.2743	8.0902	11.2811	14.8073	18.6394	22.7548	27.1351	31.7652	36.6322
.04	.0080	1.0606	2.9137	5.3004	8.1203	11.3148	14.8442	18.6792	22.7973	27.1802	31.8127	36.6820
.05	.0112	1.0759	2.9352	5.3266	8.1505	11.3485	14.8810	18.7190	22.8399	27.2253	31.8602	36.7319
.06	.0147	1.0913	2.9567	5.3528	8.1807	11.3822	14.9179	18.7589	22.8825	27.2705	31.9078	36.7818
.07	.0185	1.1068	2.9782	5.3791	8.2109	11.4160	14.9549	18.7988	22.9251	27.3156	31.9554	36.8317
.08	.0226	1.1224	2.9998	5.4054	8.2412	11.4497	14.9919	18.8387	22.9677	27.3608	32.0133	36.8816
.09	.0270	1.1380	3.0215	5.4317	8.2715	11.4836	15.0289	18.8786	23.0103	27.4060	32.0506	36.9315
0.10	0.0316	1.1537	3.0432	5.4581	8.3019	11.5174	15.0659	18.9185	23.0530	27.4512	32.0983	36.9815
.11	.0365	1.1695	3.0650	5.4845	8.3323	11.5513	15.1030	18.9585	23.0957	27.4965	32.1460	37.0315
.12	.0416	1.1853	3.0868	5.5110	8.3627	11.5852	15.1400	18.9985	23.1384	27.5418	32.1937	37.0815
.13	.0469	1.2012	3.1086	5.5375	8.3932	11.6192	15.1772	19.0386	23.1812	27.5871	32.2414	37.1315
.14	.0524	1.2172	3.1306	5.5641	8.4237	11.6532	15.2143	19.0786	23.2240	27.6324	32.2892	37.1816
.15	.0581	1.2332	3.1525	5.5907	8.4542	11.6872	15.2515	19.1187	23.2668	27.6778	32.3370	37.2317
.16	.0640	1.2494	3.1745	5.6173	8.4848	11.7213	15.2887	19.1589	23.3096	27.7232	32.3848	37.2817
.17	.0701	1.2656	3.1966	5.6440	8.5154	11.7554	15.3260	19.1990	23.3525	27.7686	32.4326	37.3319
.18	.0764	1.2818	3.2187	5.6708	8.5460	11.7895	15.3632	19.2392	23.3954	27.8140	32.4804	37.3820
.19	.0828	1.2981	3.2409	5.6975	8.5767	11.8236	15.4005	19.2794	23.4383	27.8595	32.5283	37.4322
0.20	0.0894	1.3145	3.2631	5.7243	8.6074	11.8578	15.4379	19.3196	23.4812	27.9050	32.5762	37.4824
.21	.0962	1.3310	3.2854	5.7512	8.6382	11.8920	15.4752	19.3599	23.5242	27.9514	32.6241	37.5326
.22	.1032	1.3475	3.3077	5.7781	8.6690	11.9263	15.5126	19.4002	23.5672	27.9960	32.6720	37.5828
.23	.1103	1.3641	3.3301	5.8050	8.6998	11.9606	15.5501	19.4405	23.6102	28.0416	32.7200	37.6331
.24	.1176	1.3808	3.3525	5.8320	8.7307	11.9949	15.5866	19.4808	23.6533	28.0872	32.7680	37.6833
.25	.1250	1.3975	3.3750	5.8590	8.7616	12.0293	15.6250	19.5212	23.6963	28.1328	32.8160	37.7336
.26	.1326	1.4141	3.3975	5.8861	8.7925	12.0636	15.6616	19.5576	23.7394	28.1784	32.8640	37.7840
.27	.1403	1.4312	3.4201	5.9132	8.8235	12.0981	15.7001	19.6021	23.7825	28.2241	32.9121	37.8343
.28	.1482	1.4482	3.4427	5.9403	8.8545	12.1325	15.7376	19.6425	23.8257	28.2698	32.9600	37.8847
.29	.1562	1.4652	3.4654	5.9675	8.8856	12.1670	15.7752	19.6830	23.8689	28.3155	33.0083	37.9351
0.30	0.1643	1.4822	3.4881	5.9947	8.9167	12.2015	15.8129	19.7235	23.9121	28.3612	33.0564	37.9855
.31	.1726	1.4994	3.5109	6.0220	8.9468	12.2361	15.8505	19.7641	23.9553	28.4069	33.1046	38.0359
.32	.1810	1.5166	3.5337	6.0493	8.9790	12.2706	15.8882	19.8046	23.9986	28.4527	33.1527	38.0864
.33	.1896	1.5338	3.5566	6.0767	9.0102	12.3053	15.9260	19.8452	24.0418	28.4985	33.2009	38.1369
.34	.1983	1.5512	3.5795	6.1041	9.0414	12.3399	15.9637	19.8858	24.0851	28.5444	33.2492	38.1874
.35	.2071	1.5686	3.6025	6.1315	9.0726	12.3746	16.0015	19.9265	24.1285	28.5902	33.2974	38.2379
.36	.2160	1.5860	3.6255	6.1590	9.1040	12.4093	16.0393	19.9672	24.1718	28.6361	33.3457	38.2884
.37	.2251	1.6035	3.6486	6.1865	9.1353	12.4440	16.0772	20.0079	24.2152	28.6820	33.3940	38.3390
.38	.2342	1.6211	3.6717	6.2141	9.1667	12.4788	16.1150	20.0486	24.2586	28.7279	33.4423	38.3896
.39	.2436	1.6388	3.6949	6.2417	9.1981	12.5136	16.1529	20.0894	24.3021	28.7739	33.4906	38.4402
0.40	0.2530	1.6565	3.7181	6.2693	9.2295	12.5485	16.1909	20.1302	24.3455	28.8199	33.5390	38.4908
.41	.2625	1.6743	3.7413	6.2970	9.2610	12.5833	16.2288	20.1710	24.3890	28.8659	33.5874	38.5415
.42	.2722	1.6921	3.7646	6.3247	9.2925	12.6182	16.2668	20.2118	24.4325	28.9119	33.6358	38.5922
.43	.2820	1.7100	3.7880	6.3525	9.3241	12.6532	16.3048	20.2527	24.4761	28.9579	33.6842	38.6429
.44	.2919	1.7280	3.8114	6.3803	9.3557	12.6882	16.3429	20.2936	24.5196	29.0040	33.7327	38.6936
.45	.3019	1.7460	3.8349	6.4081	9.3873	12.7232	16.3810	20.3345	24.5632	29.0501	33.7811	38.7443
.46	.3120	1.7641	3.8584	6.4360	9.4189	12.7582	16.4191	20.3755	24.6068	29.0962	33.8297	38.7951
.47	.3222	1.7823	3.8819	6.4639	9.4506	12.7933	16.4572	20.4165	24.6505	29.1424	33.8782	38.8459
.48	.3325	1.8005	3.9055	6.4919	9.4824	12.8284	16.4954	20.4575	24.6941	29.1885	33.9267	38.8967
.49	.3430	1.8188	3.9292	6.5199	9.5141	12.8635	16.5336	20.4985	24.7378	29.2347	33.9753	38.9475

Table XI. *Three-halves powers for numbers from 0 to 12.* — (Continued.)

Hundreds \ Units	0	1	2	3	4	5	6	7	8	9	10	11
0.50	0.3536	1.8371	3.9529	6.5479	9.5459	12.8986	16.5718	20.5396	24.7815	29.2810	34.0239	38.9984
.51	.3642	1.8555	3.9766	6.5760	9.5778	12.9338	16.6101	20.5807	24.8253	29.3272	34.0725	39.0493
.52	.3750	1.8740	4.0004	6.6041	9.6097	12.9691	16.6484	20.6218	24.8691	29.3735	34.1211	39.1002
.53	.3858	1.8925	4.0242	6.6323	9.6416	13.0043	16.6867	20.6630	24.9129	29.4198	34.1698	39.1511
.54	.3968	1.9111	4.0481	6.6605	9.6735	13.0396	16.7250	20.7041	24.9567	29.4661	34.2185	39.2020
.55	.4079	1.9297	4.0720	6.6887	9.7055	13.0749	16.7634	20.7453	25.0005	29.5124	34.2672	39.2530
.56	.4191	1.9484	4.0960	6.7170	9.7375	13.1103	16.8018	20.7866	25.0444	29.5588	34.3159	39.3040
.57	.4303	1.9672	4.1200	6.7453	9.7695	13.1457	16.8402	20.8278	25.0883	29.6052	34.3647	39.3550
.58	.4417	1.9860	4.1441	6.7737	9.8016	13.1811	16.8787	20.8691	25.1322	29.6516	34.4135	39.4060
.59	.4532	2.0049	4.1682	6.8021	9.8337	13.2165	16.9172	20.9104	25.1762	29.6980	34.4623	39.4571
0.60	0.4648	2.0238	4.1924	6.8305	9.8659	13.2520	16.9557	20.9518	25.2202	29.7445	34.5111	39.5082
.61	.4764	2.0429	4.2166	6.8590	9.8981	13.2875	16.9943	20.9931	25.2642	29.7910	34.5599	39.5593
.62	.4882	2.0619	4.2408	6.8875	9.9303	13.3231	17.0328	21.0345	25.3082	29.8375	34.6088	39.6104
.63	.5000	2.0810	4.2651	6.9161	9.9626	13.3587	17.0714	21.0759	25.3522	29.8841	34.6577	39.6615
.64	.5120	2.1002	4.2895	6.9447	9.9949	13.3943	17.1101	21.1174	25.3963	29.9306	34.7066	39.7127
.65	.5240	2.1195	4.3139	6.9733	10.0272	13.4299	17.1488	21.1589	25.4404	29.9772	34.7557	39.7639
.66	.5362	2.1388	4.3383	7.0020	10.0596	13.4656	17.1874	21.2004	25.4845	30.0238	34.8045	39.8151
.67	.5484	2.1581	4.3628	7.0307	10.0920	13.5013	17.2172	21.2419	25.5287	30.0704	34.8535	39.8663
.68	.5607	2.1775	4.3874	7.0595	10.1244	13.5370	17.2649	21.2834	25.5729	30.1171	34.9025	39.9176
.69	.5732	2.1970	4.4119	7.0883	10.1569	13.5728	17.3037	21.3250	25.6171	30.1638	34.9516	39.9689
0.70	0.5857	2.2165	4.4366	7.1171	10.1894	13.6086	17.3425	21.3666	25.6613	30.2105	35.0006	40.0202
.71	.5983	2.2361	4.4612	7.1460	10.2214	13.6444	17.3814	21.4083	25.7056	30.2572	35.0497	40.0715
.72	.6109	2.2558	4.4859	7.1749	10.2545	13.6803	17.4202	21.4499	25.7499	30.3040	35.0988	40.1228
.73	.6237	2.2755	4.5107	7.2038	10.2871	13.7161	17.4591	21.4916	25.7942	30.3507	35.1479	40.1742
.74	.6366	2.2952	4.5355	7.2328	10.3197	13.7521	17.4981	21.5333	25.8395	30.3975	35.1971	40.2256
.75	.6495	2.3150	4.5604	7.2618	10.3524	13.7880	17.5370	21.5751	25.8828	30.4444	35.2462	40.2770
.76	.6626	2.3349	4.5853	7.2909	10.3851	13.8240	17.5760	21.6169	25.9272	30.4912	35.2954	40.3284
.77	.6757	2.3548	4.6102	7.3200	10.4178	13.8600	17.6150	21.6587	25.9716	30.5381	35.3446	40.3798
.78	.6889	2.3748	4.6352	7.3492	10.4506	13.8961	17.6541	21.7005	26.0161	30.5850	35.3939	40.4313
.79	.7022	2.3949	4.6602	7.3783	10.4834	13.9321	17.6931	21.7423	26.0605	30.6319	35.4431	40.4828
0.80	0.7155	2.4150	4.6853	7.4076	10.5163	13.9682	17.7322	21.7842	26.1050	30.6789	35.4924	40.5343
.81	.7290	2.4351	4.7104	7.4368	10.5492	14.0044	17.7714	21.8261	26.1495	30.7258	35.5417	40.5859
.82	.7425	2.4553	4.7356	7.4661	10.5812	14.0406	17.8105	21.8681	26.1941	30.7728	35.5911	40.6374
.83	.7562	2.4756	4.7608	7.4955	10.6150	14.0768	17.8507	21.9100	26.2386	30.8198	35.6404	40.6890
.84	.7699	2.4959	4.7861	7.5248	10.6480	14.1130	17.8889	21.9520	26.2832	30.8669	35.6898	40.7406
.85	.7837	2.5163	4.8114	7.5542	10.6810	14.1493	17.9282	21.9940	26.3278	30.9139	35.7392	40.7922
.86	.7975	2.5367	4.8367	7.5837	10.7141	14.1856	17.9674	22.0361	26.3725	30.9610	35.7886	40.8439
.87	.8115	2.5572	4.8621	7.6132	10.7472	14.2219	18.0067	22.0781	26.4171	31.0081	35.8380	40.8955
.88	.8255	2.5777	4.8875	7.6427	10.7803	14.2582	18.0461	22.1202	26.4618	31.0553	35.8875	40.9472
.89	.8396	2.5983	4.9130	7.6723	10.8134	14.2946	18.0854	22.1623	26.5065	31.1024	35.9370	40.9989
0.90	0.8538	2.6190	4.9385	7.7019	10.8466	14.3311	18.1248	22.2045	26.5523	31.1496	35.9865	41.0507
.91	.8681	2.6397	4.9641	7.7315	10.8798	14.3675	18.1642	22.2467	26.5960	31.1968	36.0360	41.1024
.92	.8824	2.6604	4.9897	7.7702	10.9131	14.4040	18.2037	22.2889	26.6408	31.2441	36.0856	41.1542
.93	.8969	2.6812	5.0154	7.7909	10.9464	14.4405	18.2432	22.3311	26.6856	31.2913	36.1352	41.2060
.94	.9114	2.7021	5.0411	7.8207	10.9797	14.4770	18.2827	22.3733	26.7305	31.3386	36.1848	41.2578
.95	.9259	2.7230	5.0668	7.8505	11.0131	14.5136	18.3222	22.4156	26.7753	31.3850	36.2344	41.3097
.96	.9406	2.7440	5.0926	7.8803	11.0464	14.5502	18.3617	22.4579	26.8202	31.4332	36.2841	41.3615
.97	.9553	2.7650	5.1184	7.9102	11.0799	14.5869	18.4013	22.5003	26.8651	31.4806	36.3337	41.4134
.98	.9702	2.7861	5.1443	7.9401	11.1133	14.6235	18.4409	22.5426	26.9100	31.5280	36.3834	41.4653
.99	.9850	2.8072	5.1702	7.9700	11.1468	14.6602	18.4806	22 5850	26.9550	31.5754	36.4331	41.5173
1.00	1.0000	2.8284	5.1962	8.0000	11.1803	14.6969	18.5203	22.6274	27.0000	31.6228	36.4829	41.5692

Table XII. *Discharge in second-feet per foot of crest over sharp-crested rectangular weirs without end contractions.*[a]

D \ P	2	4	6	8	10	20	30	D \ P	2	4	6	8	10	20	30
0.1	0.13	0.13	0.13	0.13	0.13	0.13	0.13	5.1	48.25	44.09	41.96	40.73	39.97	38.45	38.03
.2	.33	.33	.33	.33	.33	.33	.33	5.2	49.79	45.50	43.29	42.01	41.20	39.61	39.17
.3	.58	.58	.58	.58	.58	.58	.58	5.3	51.36	46.93	44.64	43.30	42.45	40.78	40.31
.4	.88	.88	.87	.87	.87	.87	.87	5.4	52.94	48.38	46.00	44.60	43.71	41.96	41.47
.5	1.23	1.21	1.21	1.21	1.21	1.20	1.20	5.5	54.54	49.85	47.38	45.93	45.00	43.16	42.64
.6	1.62	1.59	1.58	1.58	1.57	1.57	1.57	5.6	56.15	51.34	48.79	47.27	46.31	44.38	43.83
.7	2.04	1.99	1.98	1.98	1.97	1.97	1.97	5.7	57.78	52.83	50.19	48.62	47.62	45.60	45.02
.8	2.50	2.43	2.41	2.41	2.40	2.40	2.40	5.8	59.42	54.34	51.62	49.99	48.94	46.83	46.22
.9	3.00	2.90	2.88	2.86	2.86	2.85	2.85	5.9	61.09	55.88	53.07	51.38	50.29	48.08	47.44
1.0	3.53	3.40	3.36	3.35	3.34	3.33	3.33	6.0	62.77	57.43	54.53	52.78	51.64	49.34	48.67
1.1	4.10	3.93	3.88	3.86	3.85	3.84	3.83	6.1	64.46	59.00	56.00	54.20	53.02	50.61	49.91
1.2	4.69	4.48	4.42	4.40	4.38	4.36	4.36	6.2	66.18	60.58	57.50	55.63	54.40	51.90	51.16
1.3	5.32	5.07	4.99	4.96	4.94	4.92	4.91	6.3	67.91	62.18	59.01	57.07	55.80	53.20	52.42
1.4	5.99	5.68	5.58	5.55	5.52	5.49	5.48	6.4	69.65	63.79	60.53	58.53	57.22	54.50	53.70
1.5	6.69	6.30	6.20	6.16	6.13	6.08	6.07	6.5	71.42	65.42	62.07	60.01	58.65	55.82	54.98
1.6	7.40	6.97	6.84	6.78	6.75	6.69	6.68	6.6	73.19	67.07	63.63	61.50	60.09	57.16	56.27
1.7	8.15	7.66	7.50	7.43	7.39	7.33	7.31	6.7	74.99	68.74	65.20	63.00	61.55	58.50	57.58
1.8	8.93	8.37	8.18	8.09	8.05	7.98	7.96	6.8	76.80	70.42	66.78	64.53	63.02	59.96	58.90
1.9	9.74	9.11	8.89	8.79	8.74	8.65	8.63	6.9	78.62	72.11	68.38	66.06	64.50	61.23	60.22
2.0	10.58	9.87	9.62	9.51	9.44	9.34	9.32	7.0	80.46	73.82	70.00	67.60	66.00	62.61	61.56
2.1	11.44	10.65	10.37	10.24	10.17	10.05	10.02	7.1	82.32	75.55	71.63	69.17	67.52	64.00	62.91
2.2	12.33	11.46	11.14	10.99	10.91	10.78	10.75	7.2	84.18	77.29	73.28	70.74	69.04	65.40	64.27
2.3	13.25	12.29	11.93	11.77	11.67	11.52	11.48	7.3	86.07	79.04	74.94	72.34	70.58	66.81	65.64
2.4	14.20	13.15	12.75	12.56	12.45	12.28	12.24	7.4	87.97	80.81	76.61	73.94	72.14	68.24	67.02
2.5	15.18	14.03	13.59	13.37	13.25	13.06	13.02	7.5	89.89	82.60	78.30	75.56	73.70	69.68	68.41
2.6	16.17	14.92	14.44	14.20	14.07	13.85	13.80	7.6	91.82	84.40	80.01	77.19	75.28	71.13	69.81
2.7	17.19	15.84	15.31	15.05	14.90	14.65	14.60	7.7	93.76	86.22	81.73	78.84	76.88	72.59	71.23
2.8	18.23	16.79	16.21	15.92	15.76	15.48	15.42	7.8	95.72	88 05	83.46	80.50	78.48	74.06	72.65
2.9	19.29	17.75	17.12	16.81	16.63	16.32	16.25	7.9	97.70	89.90	85.21	82.18	80.11	75.55	74.09
3.0	20.38	18.74	18.06	17.71	17.52	17.18	17.10	8.0	99.68	91.75	86.97	83.87	81.74	77.04	75.53
3.1	21.50	19.74	19.01	18.64	18.42	18.05	17.96	8.1	101.69	93.63	88.75	85.57	83.39	78.55	76.98
3.2	22.64	20.77	19.98	19.58	19.34	18.93	18.83	8.2	103.70	95.51	90.54	87.29	85.25	80.06	78.44
3.3	23.80	21.82	20.98	20.54	20.28	19.83	19.72	8.3	105.73	97.42	92.34	89.02	86.72	81.59	79.92
3.4	24.98	22.89	21.99	21.52	21.24	20.75	20.63	8.4	107.78	99.34	94.16	90.76	88.41	83.13	81.40
3.5	26.20	23.98	23.01	22.51	22.22	21.69	21.55	8.5	109.84	101.27	96.00	92.52	90.11	84.69	82.90
3.6	27.42	25.09	24.06	23.52	23.20	22.62	22.48	8.6	111.91	103.21	97.84	94.29	91.82	86.25	84.41
3.7	28.67	26.23	25.13	24.55	24.21	23.58	23.43	8.7	113.99	105.17	99.70	96.07	93.55	87.82	85.92
3.8	29.94	27.38	26.22	25.60	25.23	24.56	24.39	8.8	116.09	107.14	101.57	97.87	95.28	89.40	87.44
3.9	31.23	28.55	27.32	26.66	26.27	25.54	25.37	8.9	118.20	109.13	103.46	99.68	97.04	91.00	88.98
4.0	32.54	29.74	28.45	27.74	27.32	26.55	26.35	9.0	120.33	111.13	105.36	101.50	98.80	92.61	90.52
4.1	33.87	30.96	29.59	28.84	28.39	27.56	27.34	9.1	122.47	113.15	107.28	103.34	100.58	94.23	92.08
4.2	35.22	32.18	30.75	29.96	29.48	28.59	28.35	9.2	124.62	115.18	109.21	105.19	102.37	95.86	93.65
4.3	36.59	33.43	31.92	31.09	30.58	29.63	29.38	9.3	120.79	117.22	111.15	107.06	104.17	97.49	95.22
4.4	37.99	34.70	33.12	32.24	31.70	30.68	30.42	9.4	128.97	119.27	113.10	108.93	105.99	99.14	96.80
4.5	39.40	35.98	34.33	33.40	32.83	31.74	31.47	9.5	131.16	121.34	115.07	110.82	107.82	100.80	98.40
4.6	40.83	37.29	35.56	34.58	33.98	32.82	32.53	9.6	133.36	123.42	117.05	112.72	109.65	102.48	100.00
4.7	42.28	38.61	36.80	35.78	35.14	33.92	33.61	9.7	135.58	125.51	119.04	114.64	111.50	104.16	101.62
4.8	43.75	39.96	38.07	37.00	36.32	35.04	34.70	9.8	137.82	127.63	121.05	116.57	113.37	105.85	103.25
4.9	45.23	41.32	39.35	38.23	37.52	36.17	35.86	9.9	140.06	129.74	123.07	118.51	115.25	107.56	104.88
5.0	46.73	42.69	40.65	39.48	38.74	37.21	36.91	10.0	142.31	131.87	125.10	120.46	117.14	109.27	106.52

[a] This table should not be used where water on the downstream side of the weir is above the level of the crest, nor unless air circulates freely between the overfalling sheet and the downstream face of the weir. If a vacuum forms under the falling sheet the discharge may be 5 per cent greater than given in this table.

Table XIII. ***Multipliers to be used in connection with Table XII to obtain the discharge over broad-crested weirs of rectangular cross-section of type a, fig. 15.***

[P = Height of weir; c = width of crest; D = observed head, all in feet.]

D \ P / c	4.6 / 2.6	4.6 / 6.6	11.25 / .48	11.25 / .93	11.25 / 1.65	11.25 / 3.17	11.25 / 5.88	11.25 / 8.98	11.25 / 12.24	11.25 / 16.30
0.5	—	—	0.821	0.792	0.806	0.792	0.799	0.801	0.786	0.790
1.0	0.765	0.708	.997	.899	.808	.795	.791	.794	.815	.790
1.5	.789	.709	1.00	.982	.878	.796	.796	.793	.814	.792
2.0	.814	.710	1.00	1.00	.906	.815	.797	.792	.797	.793
2.5	.835	.711	1.00	1.00	.985	.844	.797	.790	.796	.793
3.0	.857	.711	1.00	1.00	1.00	.870	.797	.788	.794	.791
3.5	.878	.712	1.00	1.00	1.00	.90	.812	.787	.794	.791
4.0	.899	.714	1.00	1.00	1.00	.93	.834	.786	.792	.789
5.0	.940	.716	1.00	1.00	1.00	.97	[a]	.78	.79	.78
6.0	.986	.718	1.00	1.00	1.00	.98	[a]	.78	.78	.78
7.0	—	—	1.00	1.00	1.00	[a]	[a]	.77	.78	.77
8.0	—	—	1.00	1.00	1.00	[a]	[a]	.77	.77	.77
9.0	—	—	1.00	1.00	1.00	[a]	[a]	.77	.77	.77
10.0	—	—	1.00	1.00	1.00	[a]	[a]	.77	.77	.77

[a] Value doubtful.

Table XIV. ***Multipliers to be used in connection with Table XII to obtain the discharge over broad-crested weirs of trapezoidal cross-section of types b and c, fig. 15.***

[P = Height of weir, in feet; c = width of crest, in feet; s = upstream slope; s' = downstream slope; D = observed head, in feet.]

	Type *b*, fig. 15.							Type *c*, fig. 15.	
P	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.65	11.25
c	.33	.66	.66	.66	.66	.33	.66	7.00	6.00
s	2:1	2:1	3:1	4:1	5:1	2:1	2:1	4.67:1	6:1
s'	0	0	0	0	0	5:1	2:1	—	—
D									
1.0	1.137	1.048	1.066	1.039	1.009	1.095	1.071	1.042	1.060
1.5	1.131	1.068	1.066	1.039	1.009	1.071	1.066	1.033	1.069
2.0	1.120	1.080	1.061	1.033	1.005	1.044	1.053	1.024	1.054
2.5	1.106	1.085	1.052	1.026	.997	1.024	1.047	1.012	1.012
3.0	1.094	1.088	1.047	1.020	.991	1.009	1.047	.995	.985
3.5	1.085	1.087	1.043	1.017	.988	1.003	1.050	.983	.979
4.0	1.072	1.084	1.038	1.012	.984	1.014	1.052	.977	.976
4.5	1.064	1.081	1.035	1.009	.980	1.023	1.055	.974	.973
5.0	—	—	—	—	—	—	—	.97	.97
6.0	—	—	—	—	—	—	—	.97	.96
7.0	—	—	—	—	—	—	—	.97	.96
8.0	—	—	—	—	—	—	—	.96	.95
9.0	—	—	—	—	—	—	—	.96	.95
10.0	—	—	—	—	—	—	—	.96	.95

Tables XIII, XIV and XV give multipliers to be applied to quantitities in Table XII to determine the discharge over broad crested weirs of crest shape and height shown at the head of each column. See fig. 15.

Example: Suppose the discharge over a rectangular weir that is 10 feet long, 12 feet high, 6 feet crest width and has an observed head of 2.4 feet, is to be computed.

Table XII shows that for a height P of 12 feet and a head of $D = 2.4$, the discharge Q is 12.42 second-feet.

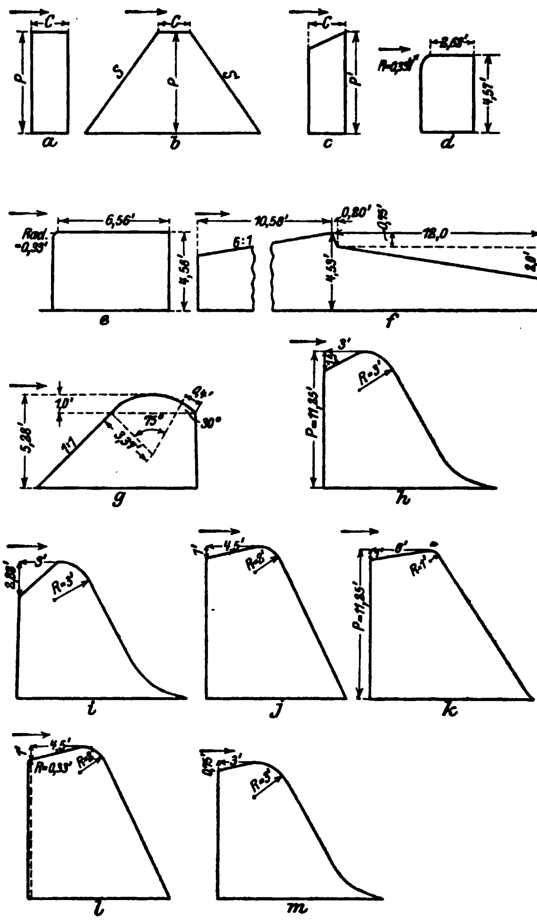


Fig. 15. Type of Weirs referred to in tables XIII, XIV, XV.

Table XIII shows that for a height P of 12 feet, a crest width c of 6 feet, and head D of 2.4 feet the multiplier is 0.797 Hence the discharge is

$$12.42 \times 0.797 \times 10 = 99.0$$

second-feet.

With two end contractions the discharge would be

$$9.9 \left(10 - \frac{2 \times 2.4}{10}\right) = 94.2.$$

Figure 16 shows the curve of discharge over the Schaghticoke dam, corrected for velocity of approach and for discharge over broad-crested weir.

Submerged weirs.

Formula of Fteley and Stearns.

$$Q = C\,L\left[H + \frac{d}{2}\right]\sqrt{Z}\,.$$

The value of the coefficient C is given in table XVI.

In this formula, Z is the difference of elevation of the upstream and downstream sides, or $Z = (H - d)$ and $d =$ measured head. Fig. 17.

Table XV. *Multipliers to be used in connection with Table XII to obtain the discharge over broad-crested weirs of compound cross-section of types d to m inclusive, fig. 15.*

[P = Height of weir, in feet; D = observed head, in feet.]

P	4.57	4.56	4.53	5.28	11.25	11.25	11.25	11.25	11.25	11.25
Type, fig. 15.	*d*	*e*	*f*	*g*	*h*	*i*	*j*	*k*	*l*	*m*
D										
0.5	—	—	—	—	0.941	0.924	0.933	0.962	0.971	0.947
1.0	0.842	0.836	0.929	0.976	1.039	1.033	.988	1.045	1.033	1.000
1.5	.866	.834	.950	.979	1.087	1.093	1.018	1.066	1.042	1.036
2.0	.888	.831	.953	.988	1.109	1.133	1.033	1.063	1.035	1.063
2.5	.906	.826	.947	1.000	1.118	1.153	1.045	1.020	1.033	1.085
3.0	.927	.822	.942	1.016	1.120	1.163	1.054	.997	1.045	1.096
3.5	.945	.817	.936	1.032	1.127	1.169	1.060	.994	1.054	1.108
4.0	.965	.812	.931	1.044	1.123	1.165	1.060	.991	1.057	1.110
5.0	1.00	.80	.92	1.05	1.11	1.16	1.05	.98	1.05	1.10
6.0	—	—	—	—	1.11	1.15	1.04	.98	1.04	1.10
7.0	—	—	—	—	1.10	1.14	1.04	.97	1.04	1.09
8.0	—	—	—	—	1.10	1.14	1.04	.97	1.03	1.09
9.0	—	—	—	—	1.09	1.14	1.03	.97	1.03	1.08
10.0	—	—	—	—	1.09	1.13	1.03	.97	1.03	1.08

Table XVI. *Fteley and Stearns's coefficients for submerged weirs.*

$\frac{d}{H}$	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	—	3.330	3.331	3.335	3 343	3.360	3.368	3.371	3.372	3.370
.1	3.365	3.359	3.352	3.343	3.335	3.327	3.318	3.310	3.302	3.294
.2	3.286	3.278	3.271	3.264	3.256	3.249	3.241	3.234	3.227	3.220
.3	3.214	3.207	3.201	3.194	3.188	3.182	3.176	3.170	3.165	3.159
.4	3.155	3.150	3.145	3.140	3.135	3.131	3.127	3.123	3.119	3.116
.5	3.113	3.110	3.107	3.104	3.102	3.100	3 098	3.096	3.095	3.093
.6	3.092	3.091	3.090	3.090	3.089	3.089	3.089	3.090	3.090	3.091
.7	3.092	3.093	3.095	3.097	3.099	3.102	3.105	3.109	3.113	3.117
.8	3.122	3.127	3.131	3.137	3.143	3.150	3.156	3.164	3.172	3.181
.9	3.190	3.200	3.209	3.221	3.233	3.247	3.262	3.280	3.300	3.325

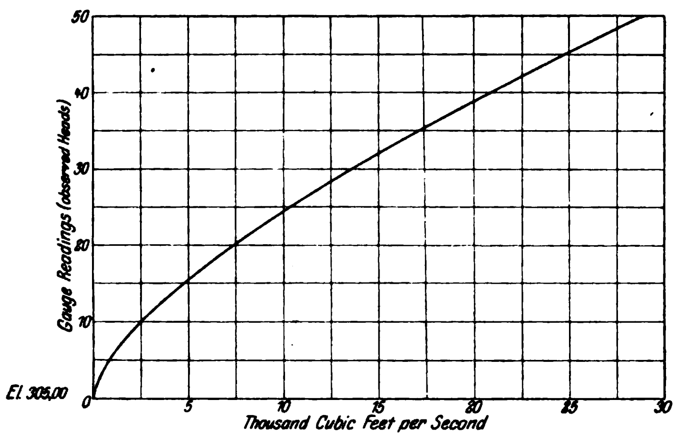


Fig. 16. Curve of Discharge over the Schaghticoke Dam.

Glancing at the table XVI it is observed that when

$$\frac{d}{H} < 0.15$$

Q is not much affected by the condition of submergence. Correction is made for velocity of approach.

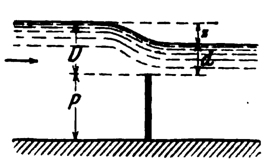


Fig. 17.

Formula of Bazin.

$$Q = m\left(1.05 + 0.21\frac{D}{P}\right)\sqrt[3]{\frac{D-d}{D}}\cdot L\,D\sqrt{2gD}$$

where m is the coefficient that would be applied to the same weir if the discharge were free, and contains correction of approach. For heads from 4 inches to 1 ft.,

$$m = 0.425 + 0.21\left(\frac{D}{P+D}\right)^2.$$

Weirs with Broad Crests.

$$Q = 2.64\,L\,H^{\frac{3}{2}}.$$

This formula, determined from experiments of the United States Geological Survey, may be applied to broad-crested weirs of any width exceeding 3 feet and heads exceeding 2 feet. The coefficient 2.64 represents 79.2 per cent of the discharge determined by the Francis formula for thin-edged weirs.

Weir curved in Plan.

A dam may be curved, or there may be an abrupt angle in plan. For curved dams, it can be assumed that the discharge would be greater than for a straight spillway very nearly in proportion to the excess in length of the arc as compared with the length of the chord.

If the dam makes an abrupt angle in plan, the discharge is computed by multiplying the quantity of flow which would be calculated by a normal dam of the same length, by the following coefficients[1]

$a=$	0^0	15^0	30^0	45^0	60^0	90^0
$f=$	0.80	0.86	0.91	0.94	0.96	1.00

where a is the value of the angle that the direction of flow makes with the dam, and $f=$ coefficient of reduction. If the dam is parallel to flow, the angle $a=0^0$ and $f=0.80$. For a dam V shaped in plan, it may be considered as formed by two dams angular in plan, which is a sufficiently close method for practice.

24. Graphic of the Francis Formula.

The particular graph which we have here under consideration is based on the ingenious method of isoplethe points, developed by M. d'Ocagne. The principle that this mathematician has established is so elegant and yet so simple, that an explanation of it will undoubtedly be appreciated, and at the same time prove the correctness of the graph of the Francis formula shown in figure 19, which has been developed by this method.

Assume a system of rectangular coordinates, as shown in figure 18. Let AB and CD be two lines parallel to the Y-axis and call their interceptions with the axis of X a and b, respectively.

Assuming two points P and Q representing a variable whose ordinates are p and q, if such two points satisfy the relation

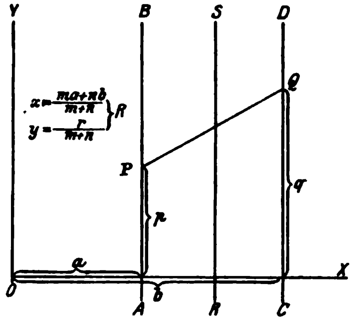


Fig. 18.

$$mp+nq=r$$

(m, n and r remaining constant), the line PQ revolves about a fixed point R, whose coordinates (x, y) can be determined without difficulty.

Effectively, R may be regarded as the point of application of the resultant of two parallel forces whose moments are

$$mp+nq=(m+n)\,y,$$
$$ma+nb=(m+n)\,x,$$

taking moments about OX and OY respectively. If we solve these two equations with respect to x and y we find

$$x=\frac{ma+nb}{(m+n)} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (1)$$

[1] M. Levy-Salvador. Hydraulique Agricole. Paris.

and

$$y = \frac{mp + nq}{(m+n)};$$

$$\text{but } mp + nq = r \text{ therefore } y = \frac{r}{(m+n)}. \quad . \quad . \quad . \quad . \ (2)$$

Expressions (1) and (2), as is readily seen, are independent of p and q, and if r alone varies, m and n remain constant.

Therefore R is a fixed point in the first case, and in the other, a point moving so that its locus is a parallel to the given lines. We may conclude then, that if we plot on the lines $A\,B$, $C\,D$, $R\,S$, points whose ordinates will be proportional respectively to

$$p, q, \frac{r}{(m+n)}$$

these values satisfying an equation of the form

$$mp + nq = r,$$

such points will be in a straight line.

Let us now apply this principle in developing the graph of the Francis formula

$$Q = 3.33\,L\,H^{\frac{3}{2}}.$$

This expression may be transformed to the logarithmic equivalent

$$\log Q = \log 3.33 + \log L + \frac{3}{2}\log H$$

which is identical with the normal form; as

$$\log Q = r;$$

$$\log L = mp;$$

$$\frac{3}{2}\log H = nq,$$

log. 3.33 is a constant, K. In this case, however, the graduation on the parallel lines will not be proportional to the variables, but to their logarithms. The work of plotting

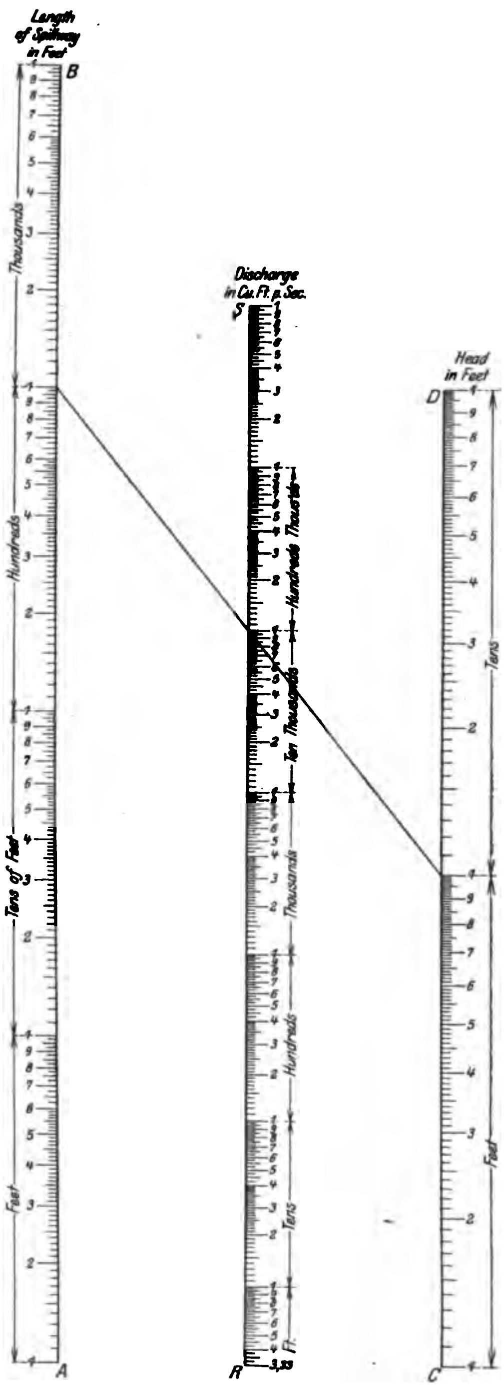


Fig. 19. Graph of Francis Formula $Q = 3.33\,LH^{\frac{3}{2}}$.

these is laid out as shown by figure 19. It is manifestly clear, that with the help of this graph two values of the Francis formula being known the other will be found readily and by a "single stroke", a thing which cannot be realized in most diagrams, or graphical representations of formulas. The rapidity and ease with which this diagram may be used will be better appreciated by its application to a practical example: The depth of water above the crest of a dam, 1000 feet long, is 10 feet, what is the discharge? Join 1000 on *A B* with 10 on *c D* and find $Q = 105{,}000$ cu. ft. per second on *RS*.

Solving in the ordinary way by the formula gives $Q = 105{,}250$ cu. ft. per second, thereby proving the accuracy of the diagram.

25. Flow Formulas for Sluice Gates.

Let $dy =$ the height of an elementary layer of an orifice,
$L =$ the width of this elementary layer,
$y =$ the distance of the elementary layer to the water level,
$H_1 =$ height of water level above upper edge of gate,
$H_2 =$ height of water level above lower edge of gate,
$Q =$ the discharge per unit time.

The theoretical discharge is expressed by

$$Q = \int_{H_1}^{H_2} \sqrt{2gy}\, L dy$$

for the flow through any orifice.

If the orifice or gate is rectangular, L is constant, and the integration gives

$$Q = \frac{2}{3} L \sqrt{2g}\left[H_2^{\frac{3}{2}} - H_1^{\frac{3}{2}}\right].$$

However, this formula is purely theoretical, and correction must be made for the influence of velocity of approach, and a contraction coefficient must be considered to allow for diminished section of gate area as the water flows through. In a series of experiments conducted by Prof. Chatterton, of the College of Engineers, Madras, India, experiments intended to determine the coefficients of discharges through the gates of some weirs on the Kistna River, it was found that these coefficients vary from 0.50 to 0.90.

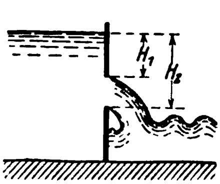


Fig. 20.

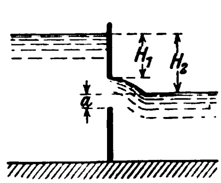


Fig. 21.

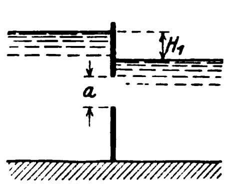


Fig. 22.

The discharge through sluice gates is influenced also by its position with reference to head and tail water.

The following formulae cover the three general cases:

for free discharge, fig. 20:

1) $$Q = \frac{2}{3} \mu L \sqrt{2g}\left[(H_2 + h)^{\frac{3}{2}} - (H_1 + h)^{\frac{3}{2}}\right]$$

if partially submerged fig. 21:

$$2)\qquad Q=\frac{2}{3}\mu L\sqrt{2g}\left[(H_2+h)^{\frac{3}{2}}-(H_1+h)^{\frac{3}{2}}\right]+\mu L a\sqrt{2g}\sqrt{H_2+h}$$

if completely submerged fig. 22:

$$3)\qquad Q=\mu L a\sqrt{2g}\sqrt{H_1+h}$$

In these equations, H_2 and H_1 have the respective values shown in the figures, and h is the velocity head or $h=\frac{v^2}{2g}$.

The value of μ is taken as 0.60 if the edge of the gate is at some distance from the river bed; if the lower edge is at level of the bed

$$\mu = 0.65 \text{ to } 0.70.$$

For circular gates, formula (3) is used, observing that $L\,a$ becomes πr^2, where $r =$ radius of the circular opening.

Chapter III. Canals.

26. Hydraulic Mean Radius, Wetted Perimeter, Slope.

The hydraulic mean radius is be represented by the expression:

$$R = \frac{A}{P}$$

or in other words, it is equal to the area of cross-section of canal or pipe, divided by that part of the perimeter of the cross-section that is under water (*wetted perimeter*). The *slope* of a channel is its natural inclination which furnishes the necessary head to the water in order to overcome the friction; it is generally denoted by the ratio of the fall to the length of the channel, and represented by the expression

$$S = \frac{h}{l}.$$

In the project of a canal, it is not necessary to keep the same slope all through, but it is a good policy to vary it according to the structure and configuration of the soil. It is well, in general to increase the slope wherever possible, as this augments the velocity, and therefore reduces the area of cross-section. The result is economy in excavation and first cost, but it must not be overlooked, that the velocity so obtained has a certain limit for soils of different nature, unless these offer strong resistance, as is the case with rock, when high velocities may be accepted. If the general formula of flow

$$v = C\sqrt{RS}$$

be considered, it is observed that the velocity, and of course the discharge, is directly proportional to the square root of the hydraulic mean radius, and the same formula, expressed thus:

$$v = C\sqrt{\frac{A}{P}S} \quad \ldots \ldots \ldots \ldots (1)$$

shows also that the discharge is inversely proportional to the square root of the wetted perimeter. It is well to note, however, that the position of the water surface, for which the mean velocity is maximum, does not correspond to the maximum of discharge. Considering expression (1) and differentiating gives:

$$av = C\sqrt{S}\,\frac{P\,dA - A\,dP}{2P^2\sqrt{\frac{A}{P}}} \quad \ldots \ldots \ldots (2)$$

The maximum of v is found by equating (2) to zero, which gives:

$$P\,d\,A - A\,d\,P = 0. \quad \ldots \ldots \ldots \quad (3)$$

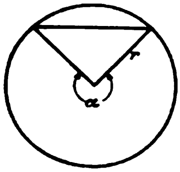


Fig. 23.

Suppose a circular cross-section as per figure 23. It is found that:

$$d\,A = \frac{r^2}{2}(1 - \cos a)\,d\,a;$$

$$d\,P = r\,d\,a$$

and equation (3) becomes: $a \cos a - \sin a = 0$; which is satisfied for $a \approx 258^0$.

27. Maximum and Minimum Water Velocities for Canals.

The velocity of the water running in a canal must be determined with due regard to the possibility of deposits being formed on the bottom, and be such, that the soil in which the canal is cut does not wash. If the water carries silt, the proper velocity is $v = 0.7$ ft. per second at least, and the silt will not be deposited; if the water carries sand the value of v must be at least 1.40 ft. per second. Table XVII gives the velocities in feet, which must not be exceeded, in order to prevent the washing of the bottom of the canal or its embankment:

Table XVII.

Nature of canal bed	Safe bottom velocity in feet per sec.	Mean velocity in feet per sec.
1. Soft brown earth . . .	0.249	0.328
2. Soft loam	0.499	0.656
3. Sand	1.000	1.312
4. Gravel	1.998	2.625
5. Pebbles	2.999	3.938
6. Broken stone, flint . .	4.003	5.570
7. Conglomerate, soft slate	4.988	6.564
8. Stratified slate	6.006	8.204
9. Hard rock	10.009	13.127

28. Distribution of Velocities.

If water is flowing through a canal, its velocity is not the same at all points of the cross-section. The friction of the molecules against each other, and against the bed and banks, has the effect of reducing considerably the discharge capacity. In general, the velocity will be less at the bed and banks than at the middle, and also if the water is shallow. See fig. 24. In designing the cross-section of a canal for given discharge conditions, it is obvious that these different velocities cannot be considered, but an average velocity figure is established, by which is meant, such a velocity as multiplied by the area of cross-section gives the discharge per second, all dimensions being in feet.

It is best, in determining the mean velocity of water for a channel, to be sure that the bottom velocity, corresponding to said average velocity, will not erode the soil. This may be obtained from Bazin's formula:

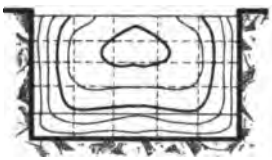


Fig. 24. Distribution of Velocities of Water in Canal of Rectangular Cross-section.

$$V_b = v - 10.87\sqrt{RS}$$

in which

v_b = bottom velocity in feet per second,
v = average velocity,
R = hydraulic radius,
S = sine of slope.

Table XVIII has been established, from this formula, and will shorten the calculations. In general, Bazin's experiments on the flow of water through channels, have shown that the loss due to friction is proportional to the wetted area and to the mean velocity, that it depends on the nature of the bed and banks, but that it is independent of the pressure of the flowing water.

29. Formulas for Canals.

The old formulas in use giving the relation between the area of cross-section, the slope and velocity of water in canals, lead to inaccuracy because of the fact that, formerly, it was considered that the degree of roughness of the bed and slopes were constant in all cases. It was determined later by Ganguillet and Kutter and also by Bazin, that the degree of roughness was highly variable, and this assumption led these investigators to develop a coefficient to cover all losses due to this and other indeterminate factors. The fundamental formula used in the design is:

$$v = C\sqrt{RS}$$

and is otherwise known as the Chezy formula. In this:

v = velocity of water in feet per second,
C = coefficient covering losses due to the degree of roughness and other factors,
R = hydraulic radius,
S = sine of slope.

Bazin gives to the coefficient C the following values:

$$C = \frac{1}{\sqrt{0.0000045\left(10.16 + \frac{1}{R}\right)}},$$ for very even surfaces, fine plastered sides and beds, planed planks;

$$C = \frac{1}{\sqrt{0.0000013\left(4.345 + \frac{1}{R}\right)}},$$ for cut-stone, brickwork, mortar, unplaned timber;

$$C = \frac{1}{\sqrt{0.00006\left(1.219 + \frac{1}{R}\right)}},$$ for slightly uneven surfaces, such as rubble masonry;

$$C = \frac{1}{\sqrt{0.00035\left(0.2438 + \frac{1}{R}\right)}},$$ for uneven surfaces, such as earth.

Bazin's coefficients give fair results for small channels of less than 20 feet bed, but Kutter's formula is used to better advantage for a wide range of channels. The value of C according to Kutter is:

$$C = \frac{41.66 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.66 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}$$

in which n is the coefficient of roughness, depending on the nature of the lining of the canal. The accuracy of the results obtained by the application of Kutter's coefficient is mainly dependent on the proper selection of the value of n; having this value, the value of C is readily determined from table XVIII for fixed values of R.

Table XVIII.

Kutter's Values of C in the formula $v = C\sqrt{RS}$ *for different values of n.*

R (feet)	Values of *n*.									
	0.009	0.010	0.011	0.012	0.013	0.015	0.017	0.020	0.025	0.030
0.1	108	94	82	73	65	53	45	35	26	20
0.2	129	113	100	89	80	66	56	45	34	26
0.3	142	124	111	99	90	75	63	52	38	30
0.4	150	132	118	106	96	80	69	56	42	34
0.5	157	139	124	111	101	85	73	60	45	36
0.6	162	143	128	116	105	89	76	63	48	38
0.7	166	147	132	119	109	92	79	65	50	40
0.8	170	151	135	122	112	95	82	68	52	42
0.9	173	154	138	125	114	97	84	70	54	43
1.00	175	156	140	127	116	99	86	71	55	45
1.20	180	160	145	131	120	103	89	74	58	47
1.40	184	164	148	135	124	106	92	77	60	49
1.60	187	167	151	137	126	108	94	79	62	51
1.80	189	169	153	140	129	110	97	81	64	53
2.00	191	172	155	142	130	112	98	83	65	54
2.50	196	176	160	146	135	116	102	86	69	57
3.00	199	179	163	149	138	119	105	89	71	59
3.50	202	182	166	152	140	122	107	91	73	61
4.00	204	184	168	154	143	124	110	93	75	63
4.50	206	186	170	156	144	126	111	95	77	64
5.00	208	188	172	158	146	127	113	97	78	66

30. Influence of Vegetation on Flow through Canals.

As has been seen, there is no hydrostatic pressure in the case of water flowing through canals, the loss of head between determined stations is nothing but the difference of level between them, or simply the inclination of the channel, which by its own nature, furnishes the necessary head to overcome friction. The velocity being affected by the slope and the cross-section, this will also be the case with the discharge. There are, of course, several factors influencing the flow or the discharge; either the character of the bed and slopes, or the alignment of the channel, its leakage, infiltration, etc. However, the prime factor that will influence the capacity of a channel is the amount of vegetation that will grow in it, vege-

tation consisting of aquatic plants, appearing after the channel has been some time in service, and which may affect the discharging capacity from one quarter to one fifth. Such plants when reaching up to the surface of the water have a preponderant influence upon the flow; they increase to no small extent the wetted perimeter, and the result is obvious. It is therefore required that much attention be given to the cleanliness of channels, but it is best also to increase by about one fifth the cross-section area at the time of establishing its dimensions.

31. Values of n in Kutter's Formula.

The following table gives the values of n generally assumed for different materials. In practice, it is better to assume a higher value of n to provide for any future contingency, such as grass growing in the canal, or possible deteriorations in the bed on account of little care being given to its preservation:

$n =$	0.009	Channel of well planed timber;
$n =$	.010	„ „ neat cement or of smooth pipes;
$n =$	.012	„ „ unplaned timber or ordinary pipe;
$n =$	.013	„ „ smooth ashlar masonry or brickwork;
$n =$	.015	„ „ ordinary brickwork;
$n =$	.017	„ „ rubble masonry;
$n =$	.020 to 0.025	Channel in earth free from obstructions;
$n =$	.030	Channel in earth, in bad order and regimen;
$n =$	.035	„ and rivers with earthen beds in bad order and regimen;
$n =$	.050	Torrents encumbered with detritus.

32. Diagram of the Ganguillet and Kutter Formula.

The diagram shown in fig. 25 is used for the graphical determination of C, n, R, S and v, in the general formula of Ganguillet and Kutter for the flow of water in rivers and other channels:

$$v = C\sqrt{R\,S} \text{ in english measure.}$$

For any case within the limits of the diagram, C, n, R and S are in a straight line drawn from the axis of abscissae through the axis of ordinates to the intersection of a slope curve and a line of roughness; viz:

n in a radial line indicating the degree of roughness at the point cut by the slope curve in question.
S in a slope curve indicating the sine of slope, at the point cut by the line of roughness in question.
C in the axis of ordinates, by the scale of C thereon.
R in the axis of abscissae, by the scale of R thereon.

Thus, any one of the four values may be found, when the others are known, by means of a straight line drawn through the points indicating the three known values.

To find v, draw a straight line from R, on the axis of abscissae, to the point on the axis of ordinates indicated by the sine of slope in question; then

draw a line parallel to the same from C on the axis or ordinates to the axis of abscissae. v is indicated by the scale for same at the point where this line cuts the axis of abscissae.

To find

$$\frac{\text{mean velocity}}{\text{max. surface velocity}} \text{ when } C \text{ is given:}$$

a straight line uniting C with the point V indicates upon the axis of abscissae the required ratio by the scale of v. In the foregoing:

$R =$ mean hydraulic radius $= \dfrac{\text{area}}{\text{wetted perimeter}}$
$S =$ sine of slope
$n =$ degree of roughness of channel
$v =$ mean velocity in feet per second
$C =$ as per paragraph 29.

33. Calculation of Channels of Rectangular Cross-section.

This is the simplest form of cross-section and easiest for calculation. The area is given by

$$A = b\,h\,.$$

The wetted perimeter by

$$P = b + 2h\,.$$

The hydraulic radius therefore is:

$$R = \frac{b\,h}{b + 2\,h}\,.$$

Chezy's formula being

$$v = C\sqrt{RS} \quad \text{or} \quad R = \frac{v^2}{C^2 S}$$

and substituting gives:

$$v = C\sqrt{\frac{b\,h\,S}{b + 2h}}\,.$$

The discharge is then found by the expression:

$$Q = b\,h\,v\,.$$

Q is a maximum when $b = 2h$; this value being found by making

$$\Theta = 90^0 \quad \text{in} \quad h = \sqrt{\frac{A \sin\Theta}{2 - \cos\Theta}}\,^{(1)}.$$

34. Calculation of a Channel Cross-section of Maximum Discharge. Trapezoidal Form.

Let $h =$ depth of water, fig. 26,
$b =$ width of bottom;
$\Theta =$ slope of embankment;
$A =$ area of channel cross-section.

[1] See paragraph No. 34.

The maximum discharge occurs when

$$h = \sqrt{\frac{A \sin \Theta}{2 - \cos \Theta}} \quad \ldots\ldots\ldots\ldots \quad (1)$$

and

$$b = \frac{A}{h} - h \operatorname{cotg} \Theta .$$

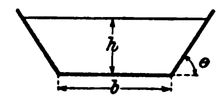


Fig. 26.

This may be demonstrated as follows:

The area is given by:

$$A = \frac{h}{2}(b + b + 2h \operatorname{cotg} \Theta)$$

$$= h(b + h \operatorname{cotg} \Theta) \quad \ldots\ldots\ldots\ldots \quad (2)$$

The wetted perimeter is then

$$P = b + \frac{2h}{\sin \Theta} \quad \ldots\ldots\ldots\ldots \quad (3)$$

The hydraulic radius is then found by:

$$\frac{P}{A} = R,$$

that is to say:

$$R = \frac{h(b + h \operatorname{cotg} \Theta)}{b + \frac{2h}{\sin \Theta}} \quad \ldots\ldots\ldots\ldots \quad (4)$$

Considering the formula:

$$v = C\sqrt{RS},$$

or this one which is equivalent:

$$Q = AC\sqrt{RS},$$

it is observed that Q is directly proportional to the square root of R, and naturally inversely proportional to the wetted perimeter, as $R = \frac{A}{P}$. The maximum of Q corresponds then to a minimum of P. From (1) and (2) above, it is deduced:

$$b = \frac{A}{h} - h \operatorname{cotg} \Theta$$

$$P = \frac{A}{h} - h \operatorname{cotg} \Theta + \frac{2h}{\sin \Theta}.$$

Differentiating and equating to zero:

$$\frac{dP}{dh} = -\frac{A}{h^2} + \frac{2 - \cos \Theta}{\sin \Theta} = 0$$

expression which, if solved for h, gives formula (1).

Table XIX contains the most advantageous values of h for different angles Θ, the area of the channel being equal to 1. The dimensions corresponding to an area A are found by multiplying the figures given in the table by $\sqrt{A}$.

Table XIX.

Θ	b	h	h cotg Θ	Width at top ($b + 2\,h$ cotg Θ)	Wetted perimeter P
90°	1.414	0.707	—	1.414	2.828
60°	0.877	0.760	0.439	1.755	2.732
45°	0.613	0.740	0.740	2.092	2.704
40°	0.525	0.722	0.860	2.246	2.771
36° 52′	0.471	0.707	0.943	2.357	2.828
35°	0.439	0.697	0.995	2.430	2.870
30°	0.336	0.664	1.150	2.656	3.012
26° 34′	0.300	0.636	1.272	2.844	3.144
1/2 Circle	—	0.798	—	1.596	2.507

Problem. It is required to design a channel of the most economical trapezoidal form, to carry 1200 cu. ft. per second.

The soil is of a flinty structure (Angle of repose $\Theta = 35^0$).

The safe bottom velocity is 4.003 feet per second, as found in table, which corresponds to a mean velocity of 5.57 feet per second.

The area of the channel cross-section is therefore:

$$\frac{1200}{5.57} = 215 \text{ sq. ft.}$$

Referring to table XIX it is seen that the wetted perimeter of most economical length is

$$2.87\,\sqrt{215} = 42.1 \text{ ft.}$$

The dimensions of the cross-section are also found in the same table

$$b = 0.439\,\sqrt{215} = 6.45 \text{ ft.}$$

$$h = 0.697\,\sqrt{215} = 10.2 \text{ ft.}$$

$$t = 2.43\,\sqrt{215} = 35.7 \text{ ft. top width.}$$

The hydraulic mean radius is therefore

$$\frac{215}{42.1} = 5.1\,.$$

The necessary slope is finally determined by the formula

$$v = C\sqrt{RS} \quad \text{or} \quad S = \frac{v^2}{C^2 R}.$$

For $\qquad n = 0.025, \quad R = 5.1, \quad C = 79, \quad C^2 = 6241.$

The slope is therefore

$$S = \frac{(5.57)^2}{6241 \times 5.1} = 0.000982$$

which corresponds to a fall of 5 feet per mile.

35. Flumes. Mechanical Aspects.

Flumes are generally built in special cases of ground depression or loose soil, where excavation for a canal is difficult, if not impossible. They have the advantage over ordinary canals in that higher velocities may be used, thereby reducing at the same time the area of cross-section.

Flumes are preferably made of reinforced concrete if permanent structures are desired.

Wooden flumes have a comparatively short life. Usually built of Oregon pine, redwood or California fir, they are of the open type, carried on trestle-work or concrete piers.

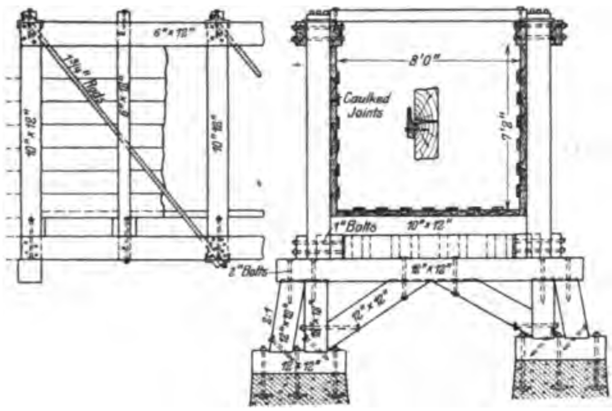


Fig. 27. Construction of Timber Flume 32 ft. span.

In designing flumes and canals it must not be overlooked that their water carrying capacity is seriously diminished in case of an ice sheet forming on the water surface. The wetted perimeter is increased in such cases. According to Frizell the value of n may be taken at 0.024 in Kutter's coefficient, the under surface of the ice being corrugated and irregular.

Fig. 27 shows the details of a timber flume with 32 ft. spans and a flume of same material but of greater width is shown in fig. 28. The latter was built by the Beaumont Irrigation Co.

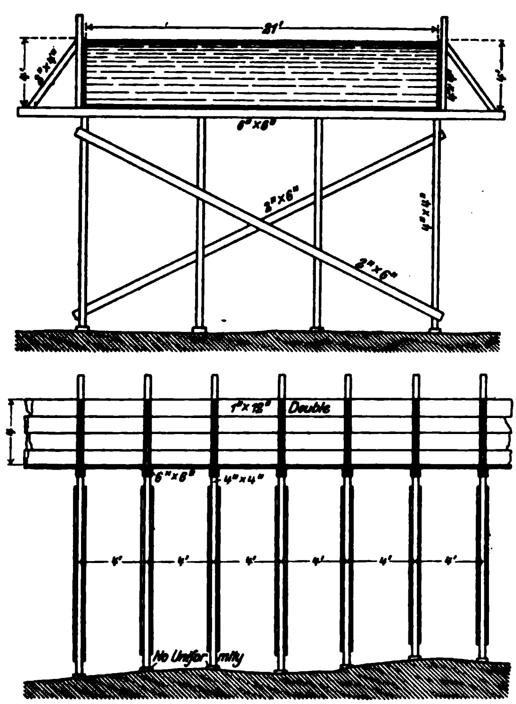


Fig. 28. Details of the Beaumont Irrigation Co's Canal, Tex.

36. Reinforced Concrete Flume.[1]

The formulae given in the detailed design below are based upon a concrete with a compressive strength of 2000 pounds per square inch and upon the use of mechanical bond reinforcing bars which have an elastic limit of 50,000 pounds per square inch; the critical amount of reinforcement under these conditions being: 0.0085 $b\ d$.

Ultimate strength:

Let $M =$ moment of resistance of section in inch lbs.

$M_0' =$ ultimate moment of resistance of section in inch lbs.

[1] From Bulletin No. 7. The Corrugated Bar Company. St. Louis, Mo.

$q =$ area of reinforcement.
$p =$ percentage of steel.
$F =$ elastic limit of steel: 50,000 lbs.

All dimensions as shown, in inches, and stresses in lbs. per sq. in.

Then we have for rectangular beams (fig. 29):

$$M_0 = 370\, b d^2 \quad \text{for} \quad q = 0.0085\, b d \quad \ldots \ldots \quad (1)$$

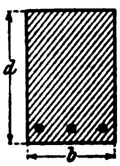


Fig. 29.

and for various percentages:

$$M_0 = 0.86\, F p b d^2 = 0.86 \times 50{,}000 \times p b d^2 \quad \ldots \quad (2)$$

$M_0 = 43{,}000\, p b d^2$ using high elastic limit corrugated bars.

Working stresses: If it is desired to design for working stresses in the steel, use the formula:

$$M = s q \times 0.86\, d \quad \ldots \ldots \quad (3)$$

where $s =$ unit stress in the steel.

Problem: Design a regular aqueduct, 4′-0″ deep and 8′-0″ wide; supported on bents at an elevation of twelve feet in the clear above grade. Depth of water: 3′-6″.

We will make the section of the aqueduct as shown in Fig. 30.

The distance center to center of the supports will be 18′-0″.

Allowable soil pressure will be taken as 3000 lbs. per sq. in.

The stress in the reinforcing steel in the sides and bottom of the aqueduct will be limited to 14,000 lbs. per sq. in., in other parts of the structure to 18,000 lbs. per sq. in.

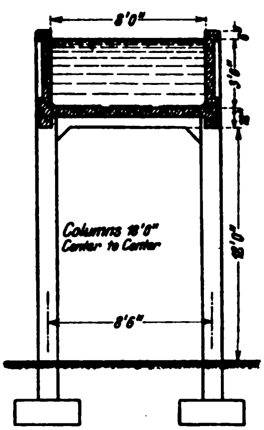


Fig. 30.

Bottom slab: The bottom slab acts as a transverse beam between the side girders. The bending moment at the center will be less than that for a free beam, owing to the partial fixity of the ends. The minimum reverse moment at the ends is equal to the overturning moment due to the horizontal pressure of the water on the sides and for a strip 12″ long $= M_e$.

$$M_e = \left(\frac{3.5 \times 62.5}{2}\right) 3.5 \times \frac{3.5}{3} = 447 \text{ ft. lbs.}$$

The maximum moment at the center of the span would be

$$M_c = \frac{w l^2 - M_e}{8}.$$

Assuming the weight of the slab to be 50 lbs,

$$w = 50 + [62.5 \times 3.5] = 269 \text{ lbs.}$$

$$M_c = 1705 \text{ ft. lbs} = 20460 \text{ in. lbs.}$$

Using the formula

$$M = s q \times 0.86\, d$$

we determine q as follows: Assume $d = 4''$

$$q = \frac{20460}{.86 \times 4 \times 14000} = 0.42 \text{ sq. in.}$$

We will make the slab 5″ thick and reinforce with $^1/_2$″ corrugated rounds spaced $5^1/_2$″ on centers. The reverse moment at the corners is:

$$M_e = 447 \times 12 = 5364 \text{ in. lbs.}$$

$$q = \frac{5364}{0.86 \times 4 \times 14000} = 0.11 \text{ sq. in.}$$

The moment at these points may be greater than M_e owing to the stiffness of the sides, we will therefore increase the amount of reinforcement as figured, and use $^3/_8$″ corrugated rounds 3′-0″ long spaced $5^1/_2$″ cts in the top of slab.

Sides: The sides act as vertical cantilevers fixed to the base, and also as girders between columns. The minimum thickness allowed will be 5″, the section to have a flange at the top to give stiffness. The overturning moment at the base equals

$$M_0 = 5364 \text{ in. lbs.}$$

The vertical reinforcement in the inside face will be $^3/_8$″ corrugated rounds spaced $5^1/_2$″ on centers. The bent-up ends of the bars in the bottom of the slab carry the moment around the corner.

Considered as girder. The load per foot run on each side is:

Water	$3.5 \times 62.5 \times 4 =$	875 lbs.
Bottom Slab	$50 \times 4 =$	200 „
Side	$150\,(0.6 \times 4.5) =$	405 „
	Total $=$	1480 lbs.

The girders will be designed as continuous beams with the same amount of reinforcement over the supports as at the center of span.

$$M = \frac{1}{12} w l^2 = 1480 \times 324 = 480.000 \text{ in. lbs.}$$

$$q = \frac{480.000}{0.86 \times 48 \times 14000} = 0.83 \text{ sq. in.}$$

Use 4 — $^1/_2$″ corrugated squares; bend two as shown in figure 32.

Shearing provisions: The vertical shear at the end of the girder $1480 \times 9 = 13320$ lbs. The average unit shear would be approximately

$$\frac{13320}{5 \times 48} = 55 \text{ lbs. per sq. in.}$$

The conrcete itself should be capable of carrying 50 lbs. of vertical shear per sq. in. and on this assumption no special provisions to carrying shearing stresses need be made. The vertical bars in the sides will act as shear members, and in order to have some vertical steel in the outside face every third bar will be put in as a U-shaped stirrup.

Temperature stresses. It is usually considered that 0.35 to 0.40 of one per cent of longitudinal reinforcement will take care of temperature and shrinkage stresses. We will use 0.35 of 1%.

The amount required in the bottom slab per foot width is:

$$0.0035 \times 12 \times 5 = 0.21 \text{ sq. in.}$$

We will use $^3/_8$″ corrugated rounds 12″ centers in top and bottom of the slab. The sides will be similarly reinforced.

Expansion joints. Every tenth bent will be a double one, and an expansion joint consisting of a bent sheet lead plate made in the flume.

Column design. The columns will be designed to resist the overturning moment due to a wind pressure of 30 lbs. per sq. ft. on the sides of the aqueduct in addition to the vertical load. The vertical load on each column is

$$18 \times 1480 = 26640 \text{ lbs.}$$

The horizontal shear in each bent due tŏ the wind is

$$18 \times 4.7 \times 30 = 2540 \text{ lbs} = H\,.$$

In the analysis for stresses due to wind action, the two columns comprising a bent will be considered as fixed in direction (by the brace beam) at the top and practically free at the bottom. (Such fixity as is given by the bases of the columns will reduce the stresses computed on a free-end assumption).

The forces acting on a bent, due to the wind load only, are shown in Figure 31. The horizontal shear in each column being taken as $^1/_2\,H$.

The value of V is found by taking the moments about the foot of one column.

$$V = \frac{2540 \times 14.5}{8.5} = 4320 \text{ lbs.}$$

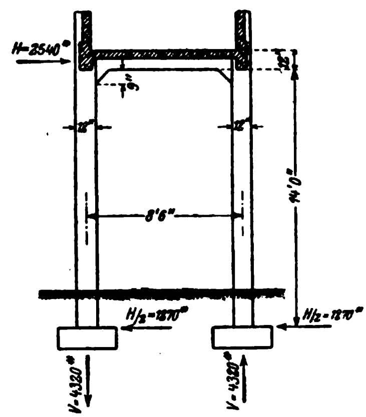


Fig. 31.

The maximumn moment in a column will be in the plane $a - a$ at the foot of the knee brace.

$$M = 1270 \times 13.25 \times 12 = 202{,}000 \text{ in. lbs.}$$

The critical section in the brace beam is at the plane $b - b$; taking moments about the center line of the beam at this plane we find the moment to be

$$M = (1270 \times 14.5) - (4320 \times 1.25) = 13015 \text{ ft. lbs.}$$
$$= 156{,}180 \text{ in. lbs.}$$

The moment in the brace beam at the center line of the bent is zero, which may be shown by taking moments about the point 0

$$M = 1270 \times 14.5 - \frac{2540 \times 14.5}{8.5} \times \frac{8.5}{2} = 0.$$

We will make the columns $12'' \times 12''$ and the brace beam $10'' \times 12''$ in section and determine the amount of reinforcement required to resist the moments developed. The knee brace will of course have to be similarly reinforced.

Column reinforcement:

$$q = \frac{202{,}000}{0.86 \times 10 \times 18{,}000} = 1.30 \text{ sq. in.}$$

This amount of steel will be required in both the outside and inside faces. Column section, $12'' \times 12''$ with $4 - {^7/_8}''$ corrugated squares, one in each corner $^1/_4''$ dia. bands, 12" centers.

Brace beam reinforcement:

$$q = \frac{156,180}{0.86 \times 10 \times 18,000} = 1.01 \text{ sq. in.}$$

Beam section $10'' \times 12''$ with $4 - {}^3/_4''$ corrugated squares.

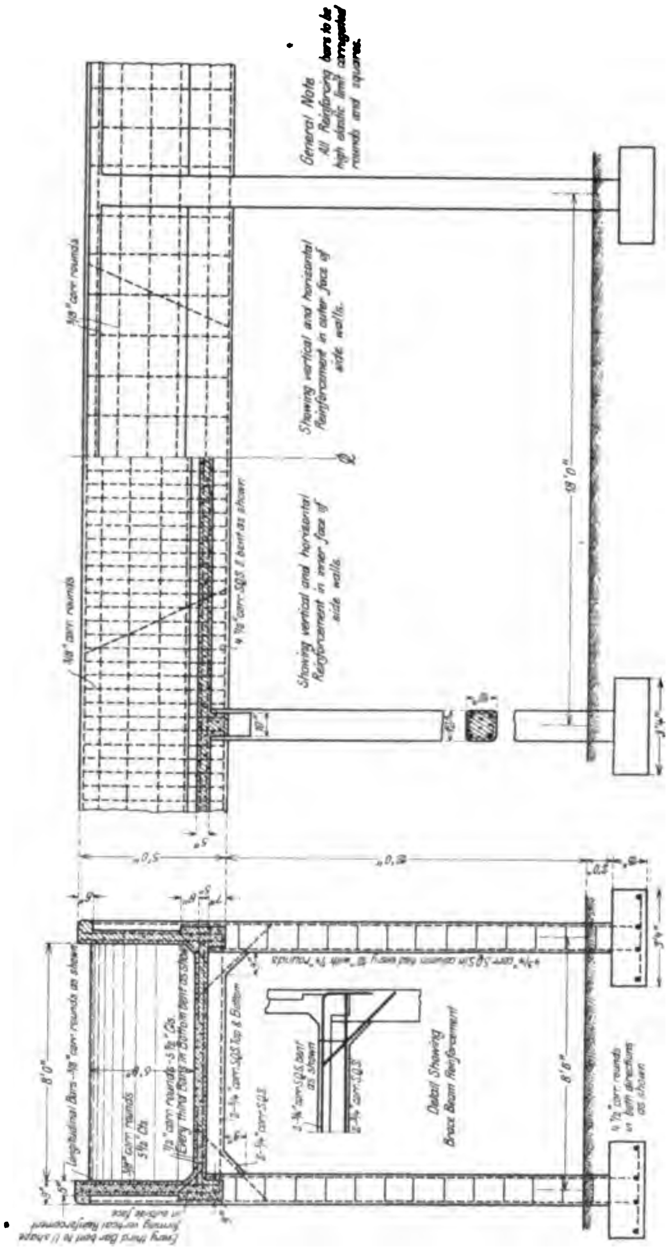


Fig. 32. Details of Reinforced Concrete Flume.

The two bars in upper face to be bent down into the column as shown in figure 32.

The reinforcement in the knee brace will be two ${}^3/_4''$ corrugated squares.

Note. If the columns were fixed in direction at the base as well as at the top a point of contra-flexure would have been developed at mid-height. The bending moments would have been reduced by one-half, the effect of fixing the base being equivalent to shortening the column by the distance from the base to the point of contra-flexure.

Stresses in the column:

Weight of structure and water per column . .	= 26,640 lbs.
Weight of column = 14 × 150	= 2,100 „
Compression due to wind	= 4,320 „
Total Maximum	= 33,060 lbs.

$$\text{Average Pressure} = \frac{33{,}060}{12 \times 12} = 230 \text{ lbs per sq. in.}$$

The maximum unit compression that can occur in the column will be on the plane $a - a$, where we have a combination of direct compression with the maximum compressive stress due to the bending moment. With this low unit stress, it will not be necessary to investigate the maximum stress on the extreme fibre.

Footings: The load on the footing is 33,060 lbs, requiring 11 square feet of bearing area at the allowed soil pressure.

Make footings 3′-4″ × 3′-4″ × 15″ deep, and reinforce with 4-$^1/_2$″ corrugated rounds each way.

Closed rectangular conduits working under head must of course be designed for the internal pressure in addition to the external loads. In the design no reliance should be placed on any possible restraining action due to the lateral pressure of the surrounding earth. For conducting water under pressure the circular type of conduit will be found preferable, except in special cases.

37. Estimated Cost of Reinforced Concrete Flume.

A concrete flume has been built by the Arizona Power Company, and its operation has resulted very successful. It was found that the friction for this form of conduit was considerably lower than for a wooden conduit of the same size, made of dressed redwood and the value of n in the Kutter formula was close to 0.0012.

The velocity of the water varies from 4 to 4.5 feet per second, the slope being 1 foot in 1,000. The quantity of water conveyed is 43 cu. ft. per sec. Considerable moss grows on the sides and bottom of the flume, and raises the water level from its normal depth of 24 in. to a depth of 34 in. The average cost of labor for constructing this flume, was divided as in table XX (next page) and corresponds to lengths of 100 feet.

Expansion joints were provided at intervals of about 100 feet, and these consisted of 1 inch boards put across the structure until the flume on either side was set. A concrete batten with cement finish top underneath the flume, a wooden batten composed of 2 × 12 planks on the outside bolted through the 1 inch joint space, form a most effective cover for the expansion joint.

Table XX.

Setting and moving forms	(14 men each $ 2)		$ 28.00
Subforeman	(1 man „ 3)		„ 3.00
Placing reinforcing metal	(3 men „ „ 2)		„ 6.00
Mixing concrete by hand	(8 „ „ „ 2)		„ 16.00
Wheeling concrete material	(2 „ „ „ 2)		„ 4.00
Wheeling mixed concrete	(5 „ „ „ 2)		„ 10.00
Shoveling into forms	(3 „ „ „ 2)		„ 6.00
Tamping	(7 „ „ „ 2)		„ 14.00
Finishing bottom	(1 man „ 2)		„ 2.00
Wetting for seasoning	(1 „ „ 2)		„ 2.00
Foreman			„ 6.00
Timekeeper			„ 3.00
		Total	$ 100.00

38. Tunnels.

The oldest tunnels were mostly of rectangular cross-section, and were not always built with a view to economy. The most practical sections are egg-shaped, or a combination of trapezoids and segments of circle.

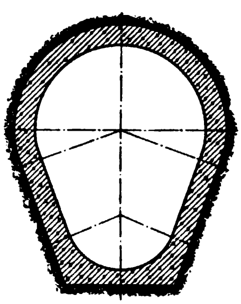


Fig. 33. Standard Cross-Section of Tunnel for constant Velocity of Water.

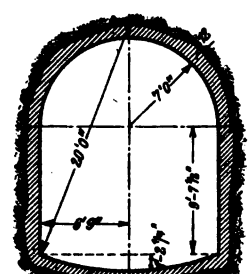


Fig. 34. Cross-section of the Great Western Power Co's Tunnel. Feather River, Cal.

If the water carries silt, and the tunnel is not under pressure, the best form of cross-section to be adopted is represented by the fig. 33. As a matter of fact, when the level of the water is lowered, the wetted area is also diminished, leaving thereby a more or less constant velocity, which, taken as a minimum to prevent the deposit of silt, is the most important factor in consideration. Of late many pressure tunnels have been constructed, where economy is justified. In order to reduce the losses due to friction, the tunnels are generally lined with concrete; experience has shown, however, that in all hard rock tunnels with arched roofs, the tendency of the rock is to break in the top centers, and the amount of concrete, that is put in place, is considerable. It is often found that a large amount of water finds its way into the tunnel, through the seams of the rock; this water usually flows under pressure.

The cross-section, fig. 34, shows the dimensions of the Great Western Power Company tunnel, which gives an approximate area of 250 square feet. This tunnel carries a quantity of water equal to about 2500 cubic feet per second at a velocity of $11^1/_2$ feet per second on a grade of 1 in 3000.

The lining was to be 12 inches thick, around the whole cross-section, permitting projections of 6 inches within this line at isolated points. It was observed, however, that the amount of lining actually averaged 21 inches.

It is necessary to make provision for exterior water pressure, if this exceeds the internal pressure. For this purpose drains are imbedded in the concrete, consisting of sections of gas pipes, which are placed through the lining for more or less each 100 square feet of area.

Care must also be taken to provide a drain by means of a pipe or other system. These drains are to leave the tunnel beneath protecting gratings. Sand traps are generally located at several places to receive the silt carried by the water. Fig. 35 shows a sand trap constructed

Fig. 35a.

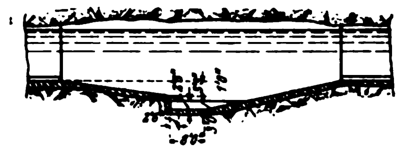


Fig. 35b.

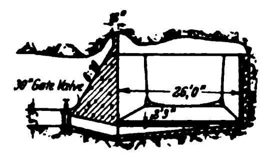


Fig. 35c.

Details of the Sand Trap in the Tunnel of the Glenwood Plant. Colo.

in the tunnel of the Glenwood hydro-electric plant of the Central Colorado Power Co. At the point where this sand trap is located, the area of the tunnel cross-section has been doubled in order to reduce the velocity. A trough has been formed across the tunnel base, which leads through a valve to a ditch in the floor.

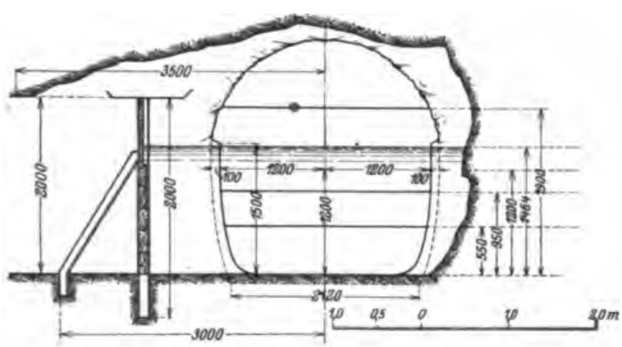


Fig. 36. Overflow Arrangement for Non-Pressure Tunnel. Brusio Plant. Switzerland.

On the down stream side of this trough a projecting slab is located in order to catch and hold all deposits of silt.

In case of seepage water encountering a non-pressure tunnel, a device consisting of an overflow weir as illustrated in fig. 36 may be installed.

39. Varied Flow.

The considerations hitherto mentioned relate to a uniform flow, that is to say, constant slope and uniform cross-section of channel. But if these conditions are not obtained, if the slope and cross-section are not of uniform nature, the flow will be varied, and calculations complicated to some extent.

The equation of varied flow is represented by

$$y_2 - y_1 = \frac{v_2^2 - v_1^2}{2g} + \int \frac{P}{A} f(v)\, dl.$$

It is to be noted, however, that the above expression is only approximate, and may be applied with better advantage under the form:

$$y_2 - y_1 = \alpha \left[\frac{v_2^2 - v_1^2}{2}\right] + \int \frac{P}{A} f(v)\, dl$$

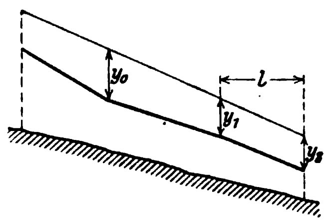


Fig. 37.

in which:

$l, y_2, y_1 =$ as per figure 37,
$v_1 =$ mean velocity at y_1,
$v_2 =$,, ,, ,, y_2,
$P =$ wetted perimeter,
$A =$ area of cross-section,
$f(v) = v$ in Kutter's formula,
$\alpha = 1.10$.

The quantity of discharge Q, being given the longitudinal profile and several transversal cross-sections, is determined by the expression

$$Q^2 = \frac{y_2 - y_1}{\frac{\alpha}{2g}\left[\frac{1}{A_2^2} - \frac{1}{A_1^2}\right] + \frac{Cl}{2}\left[\frac{P_1}{A_1^3} + \frac{P_2}{A_2^3}\right]}$$

in which:

$A_1 =$ area of cross-section at y_1
$A_2 =$ area of cross-section at y_2
$P_1 =$ wetted perimeter at y_1
$P_2 =$ wetted perimeter at y_2
$C =$ coefficient as given by Bazin (see formulas on page 44).

The problem may also be given in the reversed order. For instance, being given Q, the longitudinal and several transverse profiles, and y_2 for instance; y_1 is easily found by the above expression. Evidently any number of points may have to be determined, but the total fall is obtained by successive computations as for $y_2 - y_1$; $y_3 - y_2$; $y_4 - y_3$ etc.

Chapter IV. Pressure Pipes.

40. Flow of Water in Pipes.

The flow of water through pipes is opposed by the frictional resistance of the shell. This resistance is sometimes considerable, making it necessary to consider the corresponding loss in pipe calculations. For practical purposes, the variation in velocity of the liquid filaments flowing through a pipe is ignored; the mean velocity only enters into the calculations, this mean velocity being that which multiplied by the cross-section area of the pipe, gives the actual discharge per unit of time. The resistance offered by the shell is independent of the pressure, varies directly as the length of the pipe line, and depends on the nature of the shell. It increases also as the square of mean velocity increases, therefore it is necessary that the velocity be not too high, in order to restrict the friction loss within a certain limit.

According to Thurso, the conditions making a low speed advisable are low head, large diameter of penstock, great length of penstock, many bends in penstocks, variable loads on turbines, regulation of turbine speed by changing the amounts of water used.

Contrarily, in the case of high heads, it is permissible to use penstocks of small diameter, with few or no bends. In the latter case also, loads on the turbines must be steady, and their regulation must be taken care of.

Table XXI.

Diam. of Pressure Pipe ft.	Speed of water ft. per sec.	Diam. of Pressure Pipe ft.	Speed of water ft. per sec.
4	12.0	9	9.5
5	11.5	10	9.0
6	11.0	11	8.5
7	10.5	12	8.0
8	10.0		

Thurso has developed table XXI of high permissible speeds of water in pressure pipes of a length of 1000 feet or less; the bends being easy, and providing there are proper arrangements for the protection against water hammer. The principal losses in head of water flowing through pressure pipes are as follows:

1) *Velocity head*, represented by AB, fig. 38, is that part of the head needed to produce the velocity of water in the pipe. Its value is

$$h_v = \frac{v^2}{2g}.$$

2) ***Entry head.*** *B C*, which represents a loss occurring at the entrance of the pipe, due to the change in cross-section area, and expressed by

$$h = \left[\frac{1}{C^2} - 1\right]\frac{v^2}{2g},$$

in which C is a coefficient depending on the form of entrance. The values of C are as per table XXII.

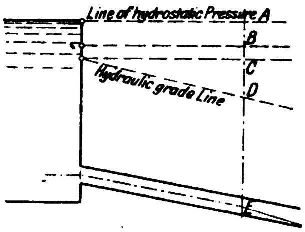


Fig. 38.

Table XXII.

Form of entrance (Fig. 39)	Value of C	$\left(\frac{1}{C^2} - 1\right)$
a	0.72	0,93
b	0.82	0,49
c or *d*	0.93 to 0.98	0.15 to 0.04

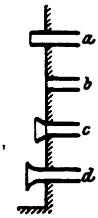


Fig. 39.

3) ***Friction head***, which is the head necessary to overcome the friction in the pipe, and is represented by *C D*.

4) ***Loss of head at bends.*** For bends of 90°, this loss is expressed by $h = n\frac{v^2}{2g}$ the values of n being as per table XXIII, the expression $\frac{r}{\mathrm{R}}$ being the ratio of the radius of the pipe r, to the radius of curvature R. Besides these losses, there is a loss of head in valves, another in the change in speed of the water, and still another due to the water leaving the lower end of the pipe. The latter is equal to the velocity head; the other two are insignificant and can be neglected in the calculations.

Table XXIII.

$\frac{v}{\mathrm{R}}$	n	$\frac{v}{\mathrm{R}}$	n	$\frac{v}{\mathrm{R}}$	n
0.1	0.13	0.5	0.29	0.8	0.98
0.2	0.14	0.6	0.44	0.9	1.41
0.3	0.16	0.7	0.66	1.0	1.98
0.4	0.21				

41. Formulas for calculating Pressure Pipes of Constant Diameter.

Kutter's formula is generally used, and although complicated in form, may be employed with tables[1] XXIV, XXV and XXVI.

Chezy's formula:

$v = C\sqrt{RS}$ with C, the Kutter coefficient, in which v, R, S are as before, gives the values expressed as follows:

$$C = \frac{v}{\sqrt{RS}}, \quad R = \frac{v^2}{C^2 S}, \quad S = \frac{v^2}{C^2 R}.$$

It will be noted that R, the hydraulic radius, is $\frac{1}{2}$ of the radius of the pipe.

The discharge capacity of a pipe may also be expressed as follows:

$$Q = Av = AC\sqrt{RS}$$

in which A is the area of cross-section of the pipe.

[1] Kent. Mechanical Engineers' Pocket Book.

Table XXIV. *Values of the factor $AC\sqrt{R}$ in the formula $Q = AC\sqrt{R} \times \sqrt{S}$ corresponding to different values of the coefficient of roughness, n. (Based on Kutter's formula).*

Diam. ft. in.	Value of $AC\sqrt{R}$ $n = .010$	$n = .011$	$n = .012$	$n = .013$	$n = .015$	$n = .017$
6	6.906	6.0627	5.3800	4.8216	3.9604	3.329
9	21.25	18.742	16.708	15.029	12.421	10.50
1	46.93	41.487	37.149	33.497	27.803	23.60
1 3	86.05	76.347	68.44	61.867	51.600	43.93
1 6	141.2	125.60	112.79	102.14	85.496	72.99
1 9	214.1	190.79	171.66	155.68	130.58	111.8
2	307.6	274.50	247.33	224.63	188.77	164
2 3	421.9	377.07	340.10	309.23	260.47	223.9
2 6	559.6	500.78	452.07	411.27	347.28	299.3
2 9	722.4	647.18	584.90	532.76	451.23	388.8
3	911.8	817.50	739.59	674.09	570.90	493.3
3 3	1128.9	1013.1	917.41	836.69	709.56	613.9
3 6	1374.7	1234.4	1118.6	1021.1	866.91	750.8
3 9	1652.1	1484.2	1345.9	1229.7	1045	906
4	1962.8	1764.3	1600.9	1463.9	1245.3	1080.7
4 6	2682.1	2413.3	2193	2007	1711.4	1487.3
5	3543	3191.8	2903.6	2659	2272.7	1977
5 6	4557.8	4111.9	3742.7	3429	2934.8	2557.2
6	5731.5	5176.3	4713.9	4322	3702.3	3232.5
6 6	7075.2	6394.9	5825.9	5339	4588.3	4010
7	8595.1	7774.3	7087	6510	5591.6	4893
7 6	10296	9318.3	8501.8	7814	6717	5884.2
8	12196	11044	10083	9272	7978.3	6995.3
8 6	14298	12954	11832	10889	9377.9	8226.3
9	16604	15049	13751	12663	10917	9580.7
9 6	19118	17338	15847	14597	12594	11061
10	21858	19834	18134	16709	14426	12678
10 6	24823	22534	20612	18996	16412	14434
11	28020	25444	23285	21464	18555	16333
11 6	31482	28593	26179	24139	20879	18395
12	35156	31937	29254	26981	23352	20584
12 6	39104	35529	32558	30041	26012	22938
13	43307	39358	36077	33301	28850	25451
13 6	47751	43412	39802	36752	31860	28117
14	52491	47739	43773	40432	35073	30965
14 6	57496	52308	47969	44322	38454	33975
15	62748	57103	52382	48413	42040	37147
16	74191	67557	62008	57343	49823	44073
17	86769	79050	72594	67140	58387	51669
18	100617	91711	84247	77932	67839	60067
19	115769	105570	96991	89759	78201	69301
20	132133	120570	110905	102559	89423	79259

For pipes of short lengths, Weisbach gives the formula

$$h = \left(0.0144 + \frac{0.01716}{\sqrt{v}}\right) \frac{l}{D} \cdot \frac{v^2}{2g}$$

in which h = loss of head;
l = length of pipe, in feet;
D = diameter of pipe, in feet;
v = velocity of water, in feet per second.

Wm. Cox deduces this formula, giving almost identical results as the Weisbach formula:

$$h = \frac{l}{D}\left(\frac{4v^2 + 5v - 2}{1200}\right)$$

in which h, l, D, and v have the same meaning as before.

Table XXV. *Table giving fall in feet per mile, the distance on slope corresponding to a fall of 1 ft., and also the values of s and $\sqrt{s}$ for use in the formula $v = C\sqrt{RS}$.*

Fall in Feet per Mile.	Slope 1 Foot in	Sine of Slope S	$\sqrt{S}$	Fall in Feet per Mile.	Slope 1 Foot in	Sine of Slope S	$\sqrt{S}$
0.25	21120	0.0000473	0.006881	17	310.6	0.0032197	0.056742
.30	17600	.0000568	.007538	18	293.3	.0034091	.058388
.40	13200	.0000758	.008704	19	277.9	.0035985	.059988
.50	10560	.0000947	.009731	20	264	.0037879	.061546
.60	8800	.0001136	.010660	22	240	.0041667	.064549
.702	7520	.0001330	.011532	24	220	.0045455	.067419
.805	6560	.0001524	.012347	26	203.1	.0049242	.070173
.904	5840	.0001712	.013085	28	188.6	.0053030	.072822
1	5280	.0001894	.013762	30	176	.0056318	.075378
1.25	4224	.0002367	.015386	35.2	150	.0066667	.081650
1.5	3520	.0002841	.016854	40	132	.0075758	.087039
1.75	3017	.0003314	.018205	44	120	.0083333	.091287
2	2640	.0003788	.019463	48	110	.0090909	.095346
2.25	2347	.0004261	.020641	52.8	100	.010	.1
2.5	2112	.0004735	.021760	60	88	.0113636	.1066
2.75	1920	.0005208	.022822	66	80	.0125	.111803
3	1760	.0005682	.023837	70.4	75	.0133333	.115470
3.25	1625	.0006154	.024807	80	66	.0151515	.123091
3.5	1508	.0006631	.025751	88	60	.0166667	.1291
3.75	1408	.0007102	.026650	96	55	.0181818	.134839
4	1320	.0007576	.027524	105.6	50	.02	.141421
5	1056	.0009470	.030773	120	44	.0227273	.150756
6	880	.0011364	.03371	132	40	.025	.158114
7	754.3	.0013257	.036416	160	33	.0303030	.174077
8	660	.0015152	.038925	220	24	.0416667	.204124
9	586.6	.0017044	.041286	264	20	.05	.223607
10	528	.0018939	.043519	330	16	.0625	.25
11	443.6	.0020833	.045643	440	12	.0833333	.288675
12	440	.0022727	.047673	528	10	.1	.316228
13	406.1	.0024621	.04962	660	8	.125	.353553
14	377.1	.0026515	.051493	880	6	.1666667	.408248
15	352	.0028409	.0533	1056	5	.2	.447214
16	330	.0030303	.055048	1320	4	.25	.5

$S = \frac{h}{l}$ = sine of angle of slope = fall of water-surface (h), in any distance (l) divided by that distance.

A formula giving the relation of diameter of pipe to quantity discharged is Hagen's formula

$$D = C\left[\frac{Q}{\sqrt{S}}\right]^{0.387}$$

in which $C = 0.239$; D and Q being in feet and cubic feet per second respectively.

Vallot, a French engineer, makes D vary as the 0.375 power of the discharge and his formula is expressed as follows:

$$D = 0.376\left[\frac{Q}{\sqrt{S}}\right]^{3/8}$$

which is equivalent to this other

$$S = 0.0054\frac{Q^2}{D^{16/3}}.$$

It is noted from the different formulas exposed above, that the quantity of water discharged through a pipe depends on the head, this being

Table XXVI. *Values of $\sqrt{R}$ for circular pipes, sewers and conduits of different diameters.*

Diam. ft. in.	$\sqrt{R}$ in feet	Diam. ft. in.	$\sqrt{R}$ in feet	Diam. ft. in.	$\sqrt{R}$ in feet	Diam. ft. in.	$\sqrt{R}$ in feet
3/8	0.088	2	0.707	4 6	1.061	9	1.500
1/2	.102	2 1	.722	4 7	1.070	9 3	1.521
3/4	.125	2 2	.736	4 8	1.080	9 6	1.541
1	.144	2 3	.750	4 9	1.089	9 9	1.561
1 1/4	.161	2 4	.764	4 10	1.099	10	1.681
1 1/2	.177	2 5	.777	4 11	1.109	10 3	1.601
1 3/4	.191	2 6	.790	5	1.118	10 6	1.620
2	.204	2 7	.804	5 1	1.127	10 9	1.639
2 1/2	.228	2 8	.817	5 2	1.137	11	1.658
3	.251	2 9	.829	5 3	1.146	11 3	1.677
4	.290	2 10	.842	5 4	1.155	11 6	1.696
5	.323	2 11	.854	5 5	1.164	11 9	1.714
6	.354	3	.866	5 6	1.173	12	1.732
7	.382	3 1	.878	5 7	1.181	12 3	1.750
8	.408	3 2	.890	5 8	1.190	12 6	1.768
9	.433	3 3	.901	5 9	1.199	12 9	1.785
10	.456	3 4	.913	5 10	1.208	13	1.803
11	.479	3 5	.924	5 11	1.216	13 3	1.820
1	.500	3 6	.935	6	1.225	13 6	1.837
1 1	.520	3 7	.946	6 3	1.250	14	1.871
1 2	.540	3 8	.957	6 6	1.275	14 6	1.904
1 3	.559	3 9	.968	6 9	1.299	15	1.936
1 4	.577	3 10	.979	7	1.323	15 6	1.968
1 5	.595	3 11	.990	7 3	1.346	16	2.
1 6	.612	4	1.	7 6	1.369	16 6	2.031
1 7	.629	4 1	1.010	7 9	1.392	17	2.061
1 8	.646	4 2	1.021	8	1.414	17 6	2.091
1 9	.661	4 3	1.031	8 3	1.436	18	2.121
1 10	.677	4 4	1.041	8 6	1.458	19	2.180
1 11	.692	4 5	1.051	8 9	1.479	20	2.236

the vertical distance between the level surface of still water in the head race and the surface of tail race water; it depends also upon the length of the pipe, and the character of its interior surface (Kutter formula); although it is perfectly independent of the position of the pipe, whether horizontal or inclined.

At any point where the pipe line is located above the hydraulic grade line, the pressure is less than the atmospheric pressure, the result being a siphon-action, except when the elevation of the pipe above the hydraulic grade line does not exceed the height of the mercury column at the point under consideration multiplied by the density of mercury: 13.58.

42. Pressure Pipes of Variable Diameter.

If the loss of head is assumed beforehand, it is more economical to make the pipe of variable diameter.

The weight of a pipe may be represented by the following relations:

$W = f(H^2 D^2)$, for a vertical pipe,

$W = f\left(\frac{H^2 D^2}{\sin \alpha}\right)$, inclination of pipe being determined by angle α.

$W = f(2 H l D^2)$ for a horizontal pipe of length l.

H, l and D are in feet. f means functions of.

Let n be the number of sections into which H is divided, and call r the n^{th} section. When n is odd, r is a section the diameter of which would be equal to the diameter of a constant diameter pipe line[1]. The total weight of the line is determined by

$$W' = \frac{W}{n^2}\left(\frac{n+1}{2}\right)^{\frac{2}{5}} \int \left(\frac{2r-1}{r^{\frac{2}{5}}}\right)_{r=1}^{r=n} = W\mu$$

in which W is determined as above.

The coefficient of W is < 1 when $n = 2$, and the smaller it is, the more advantageous will be a pipe line of variable diameter.

It can be assumed that

$$\mu = \frac{W'}{W} = 0.95 - \frac{1}{2n};$$

0.95 being the value of μ when $n = \infty$.

The influence of a variable diameter pipe line upon the hydraulic gradient and the weight is shown in fig 40a.

Let AO be a pipe line. The hydraulic grade line in case of a constant diameter, would be the straight line BO, and the weight would be represented by the area of the triangle CXO, whereas for a pipe line of variable diameter, the hydraulic grade line will be the curve BDO, and the weight

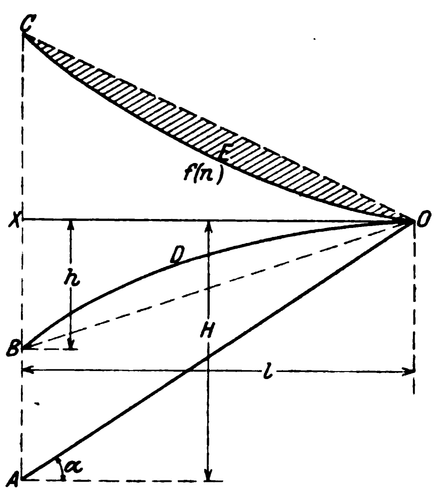


Fig. 40a.

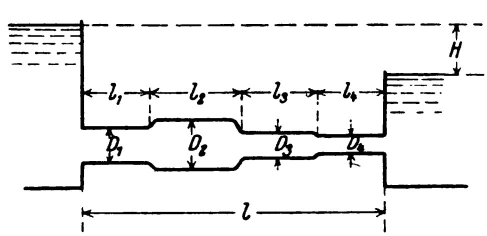


Fig. 40b.

will be represented by the area $CEOX$, showing an economy corresponding to the cross-sectioned area CEO. Trautwine gives the following formula for calculating the discharge of a variable diameter pipe line: (fig. 40b)

$$Q = \frac{\pi}{4}\sqrt{\frac{2gH}{F_1\frac{l_1}{D_1^5} + F_2\frac{l_2}{D_2^5} + \ldots + F_n\frac{l_n}{D_n^5}}}$$

in which $F_1, F_2, \ldots Fn$ are the corresponding values of the friction factor as determined by

$$h = F\frac{l}{D}\frac{v^2}{2g}$$

h being the friction head.

[1] E. Pacoret. La Houille Blanche. Paris.

Table XXVII. *Circumferences and areas of circles.*

Diam. D	Circumf. πD	Areas $\frac{\pi D^2}{4}$	Diam. D	Circumf. πD	Areas $\frac{\pi D^2}{4}$	Diam. D	Circumf. πD	Areas $\frac{\pi D^2}{4}$
1	3.1416	.7854	51	160.2212	2042.821	101	317.3009	8011.85
2	6.2832	3.1416	52	163.3628	2123.717	102	320.4425	8171.28
3	9.4248	7.0686	53	166.5044	2206.183	103	323.5840	8332.29
4	12.5664	12.5664	54	169.6460	2290.221	104	326.7256	8494.87
5	15.7080	19.6350	55	172.7876	2375.829	105	329.8672	8659.01
6	18.8496	28.2743	56	175.9292	2463.009	106	333.0088	8824.73
7	21.9912	38.4845	57	179.0708	2551.759	107	336.1504	8992.02
8	25.1327	50.2655	58	182.2124	2642.079	108	339.2920	9160.88
9	28.2743	63.6173	59	185.3540	2733.971	109	342.4336	9331.32
10	31.4159	78.5398	60	188.4956	2829.433	110	345.5752	9503.32
11	34.5575	95.0332	61	191.6372	2922.467	111	348.7168	9676.89
12	37.6991	113.097	62	194.7787	3019.071	112	351.8584	9852.03
13	40.8407	132.732	63	197.9203	3117.245	113	355.0000	10028.75
14	43.9823	153.938	64	201.0619	3216.991	114	358.1416	10207.03
15	47.1239	176.715	65	204.2035	3318.307	115	361.2832	10386.89
16	50.2655	201.062	66	207.3451	3421.194	116	364.4247	10568.32
17	53.4071	226.980	67	210.4867	3525.652	117	367.5663	10751.32
18	56.5489	254.469	68	213.6283	3631.681	118	370.7079	10935.88
19	59.6903	283.529	69	216.7699	3739.281	119	373.8495	11122.02
20	62.8318	314.159	70	219.9115	3848.451	120	376.9911	11309.73
21	65.9734	346.361	71	223.0531	3959.192	121	380.1327	11499.01
22	69.1150	380.133	72	226.1947	4071.504	122	383.2743	11689.87
23	72.2566	415.476	73	229.3363	4185.387	123	386.4159	11882.29
24	75.3982	452.389	74	232.4779	4300.840	124	389.5575	12076.28
25	78.5398	490.874	75	235.6194	4417.865	125	392.6991	12271.85
26	81.6814	530.929	76	238.7610	4536.460	126	395.8407	12468.98
27	84.8230	572.555	77	241.9026	4656.626	127	398.9823	12667.69
28	87.9646	615.752	78	245.0442	4778.362	128	402.1239	12867.96
29	91.1062	660.520	79	248.1858	4901.670	129	405.2655	13069.81
30	94.2478	706.858	80	251.3274	5026.548	130	408.4070	13273.23
31	97.3894	754.768	81	254.4690	5152.997	131	411.5486	13478.22
32	100.5310	804.248	82	257.6106	5281.017	132	414.6902	13684.78
33	103.6726	855.299	83	260.7522	5410.608	133	417.8318	13892.91
34	106.8142	907.920	84	263.8938	5541.769	134	420.9734	14102.61
35	109.9557	962.113	85	267.0354	5674.502	135	424.1150	14313.88
36	113.0973	1017.876	86	270.1770	5808.805	136	427.2566	14526.72
37	116.2389	1075.210	87	273.3186	5944.679	137	430.3982	14741.14
38	119.3805	1134.115	88	276.4602	6082.123	138	433.5398	14957.12
39	122.5221	1194.591	89	279.6017	6221.139	139	436.6814	15174.68
40	125.6637	1256.637	90	282.7433	6361.725	140	439.8230	15393.80
41	128.8053	1320.254	91	285.8849	6503.882	141	442.9646	15614.50
42	131.9469	1385.442	92	289.0265	6647.610	142	446.1062	15826.77
43	135.0885	1452.201	93	292.1681	6792.909	143	449.2477	16060.61
44	138.2301	1520.531	94	295.3097	6939.778	144	452.3893	16286.02
45	141.3717	1590.431	95	298.4513	7088.218	145	455.5309	16513.
46	144.5133	1661.903	96	301.5929	7238.229	146	458.6725	16741.55
47	147.6549	1734.945	97	304.7345	7389.811	147	461.8141	16971.67
48	150.7964	1809.557	98	307.8761	7542.964	148	464.9557	17203.36
49	153.9380	1885.741	99	311.0177	7697.687	149	468.0973	17436.62
50	157.0796	1963.495	100	314.1593	7853.982	150	471.2389	17671.46

43. Penstock Diagram.

For problems dealing with penstocks of larger sizes, the Kutter formula is much used and the required relations of its factors are found by the tables. However, in the preliminary estimates of work relating to hydraulic engineering, such as the calculations of penstocks, a hair splitting accuracy is not imperative. As the complicated character of the Kutter formula is not adaptable theoretically to the diagram which has been developed, it has been found necessary to resort to a formula much used in Europe. The problem is to tie together in a regular network the relations that determine the size of a penstock, and such other useful factors as are likely to be demanded. The theoretical basis of the diagram was developed by Com. L. Bertrand, and is nothing but an application and perfection of the method of isoplethe points devised by Prof. M. d'Ocagne. A short analysis will first be made of this interesting principle, so that a clear understanding will exist as to the mathematical accuracy of the penstock diagram subsequently developed.

Consider an equation

$$Z = a_1 y_1 + a_2 y_2 + a_3 y_3 + \dots + a_n y_n,$$

where Z is a function of the n variables $y_1, y_2 \dots y_n$ and where a_1, a_2, a_n are constants.

If in a system of co-ordinates we draw parallel ordinates taken arbitrarily, whose abscissae are $x_1, x_2, \dots x_n$ and the lengths of which are proportional to $y_1, y_2 \dots y_n$ the second member of the above equation represents the sum of the moments of parallel forces $a_1, a_2, \dots a_n$ with respect to the axis of the abscissae and applied at the extremities of the lengths $y_1, y_2 \dots y_n$.

The co-ordinates of X and Y of the point of application of the resultant force will be:

$$X = \frac{a_1 x_1 + a_2 x_2 + \dots + a_n x_n}{a_1 + a_2 + \dots a_n}$$

$$Y = \frac{a_1 y_1 + a_2 y_2 + \dots + a_n y_n}{a_1 + a_2 + \dots a_n}$$

$$= \frac{Z}{a_1 + a_2 + \dots a_n}.$$

X depends only on a and x, which are invariables, being constant by hypothesis, and x taken arbitrarily. The point of application of the resultant is, therefore, always on the same ordinate whose abscissa is determined. It is possible, then, by obtaining this point of application and measuring its ordinate with a determined scale, to find the value of Z. The geometrical determination of this point of application is similar to the fixation of the point of application of two parallel forces. The resultants (1, 2) of the two first (1) and (2) combined with the third force (3) gives the resultant (1, 2, 3) of the system of these three forces.

The resultant (1, 2, 3) combined with (4) will give the resultant (1, 2, 3, 4) of the system of four forces, and so on. It is possible to operate in any order; (2) and (3) combined will give (2, 3) and with (4) will give

(2, 3, 4) and this one combined with (1) will give the same resultant (1, 2, 3, 4) and the same point of application.

Let us take an equation of the form

$$Z_{(1,\,2)} = a_1\, y_1 + a_2\, y_2\,.$$

On the line $A\,B$, figure 41, take arbitrarily the points x_1, x_2, then determine the point corresponding to the abscissa X, which divides the distance x_1, x_2 in two parts inversely proportional to a_1 and a_2. Then draw the ordinate lines of these three points. If on the ordinate lines of x_1 and x_2 we plot lengths y_1 and y_2, and if we join the extremities (1) and (2) of these ordinates by a line, the point (1, 2) where this line cuts the ordinate of $x_{1,\,2}$ will be a point of application of the resultant of two parallel forces applied at (1) and (2) and the value of z will be measured by the

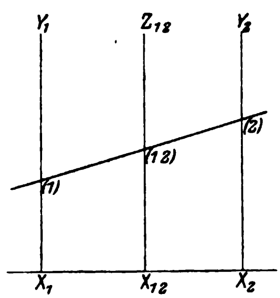


Fig. 41.

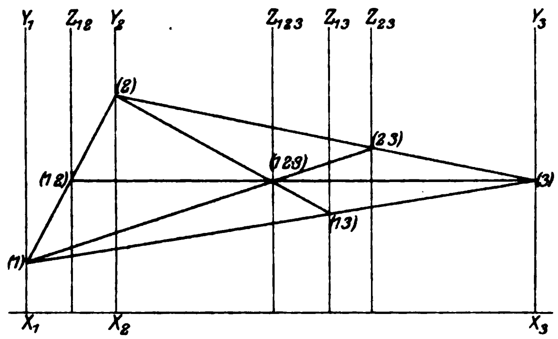


Fig. 42.

length of the ordinate (1, 2) to a determined scale. Therefore, if the three ordinates are graduated to convenient scales, it will be possible by the line (1, 2) to have the value of

$$Z_{(1,\,2)} = a_1\, y_1 + a_2\, y_2$$

by a single reading, no matter what be the values of the variables y_1 and y_2. By the same method, Fig. 42, it is possible to calculate the value of the function

$$Z_{(1,\,2,\,3)} = a_1\, y_1 + a_2\, y_2 + a_3\, y_3$$

and also the three partial combinations:

$$Z_{(2,\,3)} = a_2\, y_2 + a_3\, y_3$$
$$Z_{(1,\,3)} = a_1\, y_1 + a_3\, y_3$$
$$Z_{(1,\,2)} = a_1\, y_1 + a_2\, y_2\,.$$

It is seen that a diagram can be constructed composed of parallel lines, at the rate of one line for each variable Y, which will give not only the value of these n variables in

$$Z = a_1\, y_1 + a_2\, y_2 + \ldots + a_n\, y_n$$

but also any one of the linear functions of the same form into which this equation can be decomposed, regardless of the number of terms and their position.

A function of the form

$$v = u_1^{m_1} \times u_2^{m_2} \times \ldots \times u_n^{m_n}$$

whose second nember is a monomial form may be transformed in its equivalent, which is:

$$\log v = m_1 \log u_1 + m_2 \log u_2 + \ldots + m_n \log u_n .$$

In this case, to find graphically the values as explained above, it is necessary to graduate the ordinates in proportion to the logarithms of the variables. The whole principle is ingenious and simple as we shall show by developing a formula and a diagram for calculating penstocks.

The Vallot formula which is

$$D = 0.324 \left[\frac{Q}{\sqrt{S}}\right]^{\frac{3}{8}}$$

in metric measure, may be transformed to

$$D = 0.376 \left[\frac{Q}{\sqrt{S}}\right]^{\frac{3}{8}}$$

in english measures, and is for penstocks that have been some time in service.

Results obtained with this formula are in close accord with those of observation, especially for pipes of large diameter. In fact, experiments made with steel pipes having diameters ranging from 68 inches to 90 inches have shown the superiority of this formula over any other, so far as pipes lined with deposits are concerned.

It is seen that this formula is adaptable to logarithmic transformation giving any one of the three quantities D, Q, S in terms of the two others. Further we can find other quantities which depend on these, and determine them in the same graphical way.

Discharge Q is related to velocity v and diameter D, as shown by the equation

$$4\,Q = \pi D^2 v .$$

The total length of pipe line and fall per mile in feet are related to the slope by the expression

$$h = lS .$$

Power in function of head and discharge at 80 per cent efficiency of wheels are given by

$$P = \frac{Q \cdot H}{11} .$$

Thickness of shell of pipe, if 12,500 lbs per square inch is taken as safe tensile stress and a riveted joint of 75 per cent efficiency is considered, is given by the relation

$$\frac{D \cdot H}{3600} = t .$$

D in this expression is in feet.

The logarithmic transformation of the flow formula is:

$$\log D - \log 0.376 = \frac{3}{8} \log Q - \frac{3}{16} \log S$$

and the other transformations are made similarly.

The diagram constructed after the method we have described above is given in fig. 43 and contains 8 parallel lines which are graduated logarithmically. It will be noted that on each one of these lines, the same con-

stant distance, limited by two heavy lines, separates two numbers, one of which is ten times greater than the other, so that it corresponds to the order of decimals, units, tens, hundreds, etc., shown on the figure. As to the numbers themselves, they are read as on the slide rule.

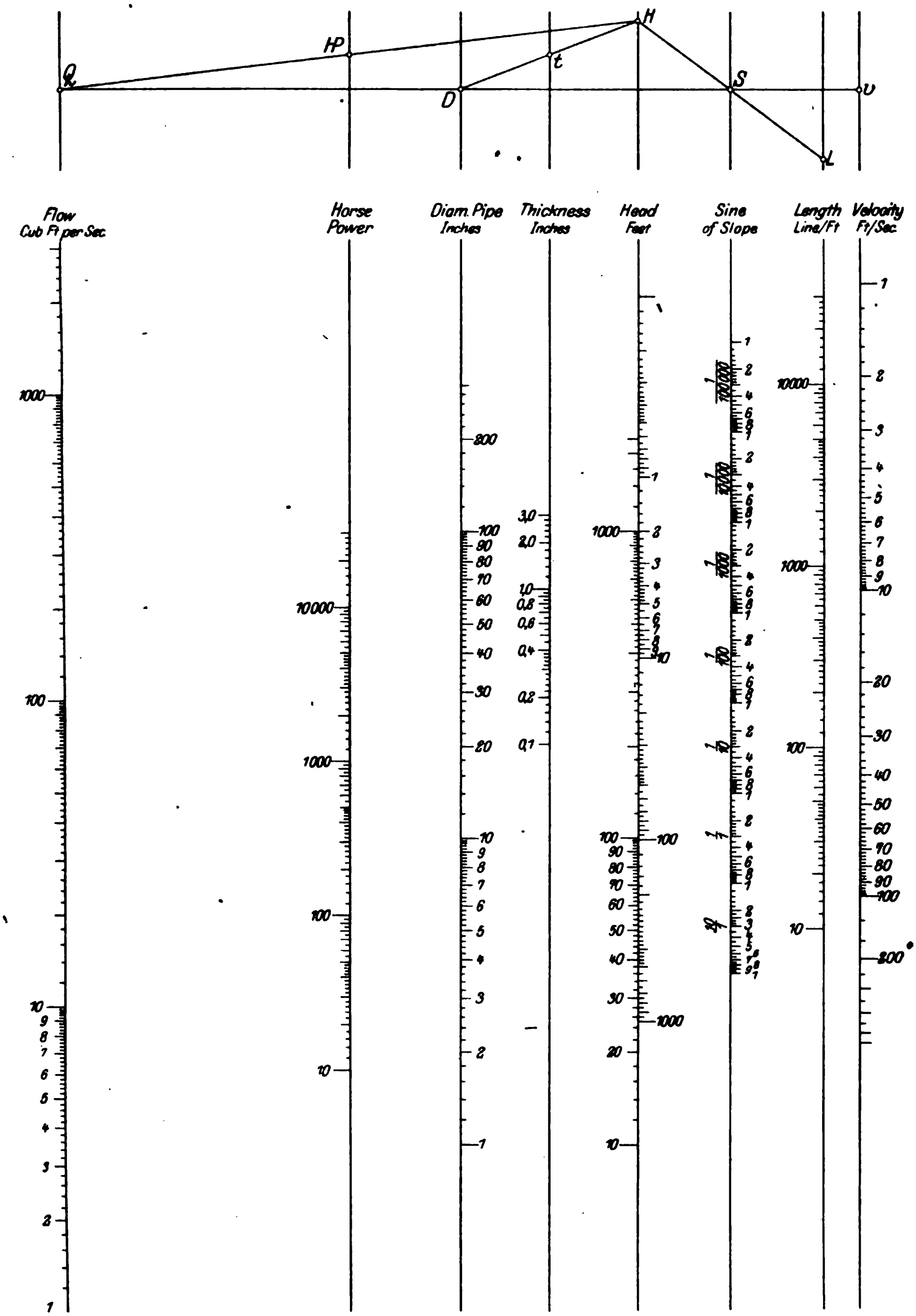


Fig. 43. The Penstock Diagram.

It may now be deduced from what has been shown that, on the same straight transversal line, the diagram gives the following groups of quantities:

First: Flow Q, diameter D, slope S, velocity v.

Second: Diameter D, head H, horse power.

Third: Total loss of head h, slope S, length l.

Fourth: Head H, diameter D, thickness of metal t.

It should be noted that H has two scales, the one on the right to be used with S and l, and the one on left with t and horse power.

Let us solve a specified problem:

It is proposed to carry 550 cu. ft. of water per second a distance of 3 miles with a loss of head of 3 feet, using a riveted steel pipe, having a joint efficiency of 75 per cent. Find all elements concerned.

The slope S is first found to be $S = 0.000189$.

Join 0.000 189 on S with 550 on Q finding, $D = 168$ in.; $v = 3.6$ ft. per second.

If this water was flowing under a pressure head of 150 feet, the thickness of metal would be found to be $t = 0.6 = \frac{5}{8}$ in. by joining 150 on H with 168 on D.

If the water is used for power purposes and must run through wheels having an efficiency of 80 per cent, a line through 147 the effective head, and 550 on Q gives 7300 actual horse power on the scale.

As an example of comparison, if it is desired to solve the first part of the problem by the Kutter formula, we find with

$$n = 0.0135\,,$$

and by means of the tables XXIV, XXV, XXVI:

for a pipe 12.6 ft. in diameter, trial size,

$$AC\sqrt{R} = 30{,}041$$

and for a fall of 1 ft. per mile

$$\sqrt{S} = 0.013762\,.$$

Therefore,

$$Q = 0.013762 \times 30{,}041 = 412$$

cubic feet per second, which is too small a flow; we next try a 14 ft. pipe, finding

$$AC\sqrt{R} = 40{,}432$$

with S as above; $Q = 556$, which is good. Therefore 168 inches given by the diagram justifies the use of the formula on which it is based.

Because of the projecting rivet heads and lap edges, Mr. Kuichling states that riveted pipe must have a diameter about 8 per cent greater than cast iron pipe for the same discharge.

The upper part of the diagram shows the combinations that can be made to solve any case.

In using this diagram, and to prevent its deterioration by the drawing of lines it is recommended to use a sheet of tracing cloth on which a line has been previously drawn.

44. Diagrams for computing Thickness of Steel Pipe Shells for different Joint Efficiencies.

The task of determining the thickness of steel pipe shells for any individual case under specified conditions is relatively easy, but when the engineer has to make careful investigations and design his pipes most economically with regard to hydrostatic pressure, the work becomes tedious, and demands much time for computing and checking. The value of the accompanying diagrams, which give immediately the thickness of shells for most cases encountered in practice, will no doubt, be recognized by hydraulic engineers and those on whom devolve the task of designing either ordinary riveted steel pipes, surge tanks for hydroelectric plants, or tanks for water storage.

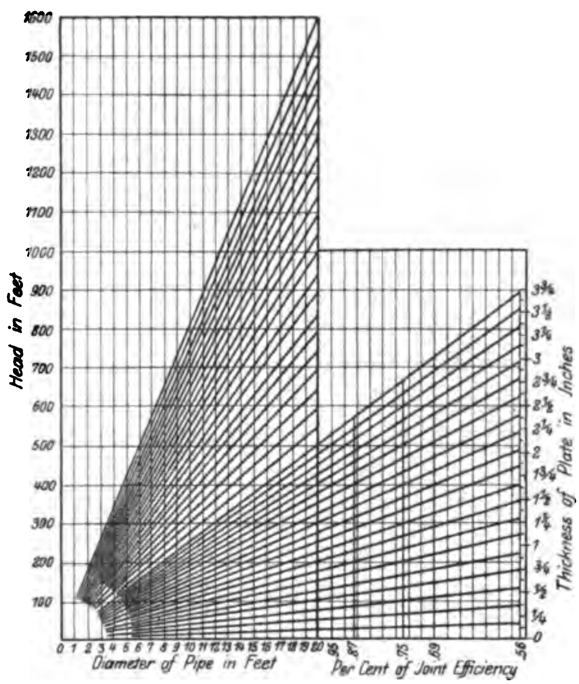


Fig. 44.

Fig. 44 has been developed so as to cover most cases liable to present themselves in water-power devlopments, under heads as high as 1600 ft.

Fig. 45 is particularly intended to be useful to designers of water towers, and with its help they will be able to determine the requirements for most cases.

The diagrams are based on the following assumptions:

Tensile strength of plate 50.000 lbs per sq. in. of section, factor of safety 4.

Percentage efficiency of riveted joints[1]):

[1] From a discussion on the theory and practice of riveting, contained in an address delivered to students of Cornell College by Mr. J. M. Allen, Pres. Hartford Steam Boiler & Insurance Co.

Single riveted joint	56
Double „ „	69
Triple „ „	75
Double-welded „	87
Quadruple-riveted „	95

To find the thickness of shell of a steel riveted pipe for head and diameter, join the origin 0 with given head on $A\ B$ and follow this line until it intersects the vertical line passing through the given diameter. At this point draw a parallel to $0\ A$ meeting $A\ B$, join again this point of intersection with the origin 0, continue to assumed efficiency line. At this point a parallel to $0\ A$ shows required thickness on scale.

If the problem is reversed, that is, to find safe head for a case in which diameter of pipe, thickness of shell and efficiency of riveted joint are given; through given thickness on scale draw a parallel to $0\ A$, meeting efficiency

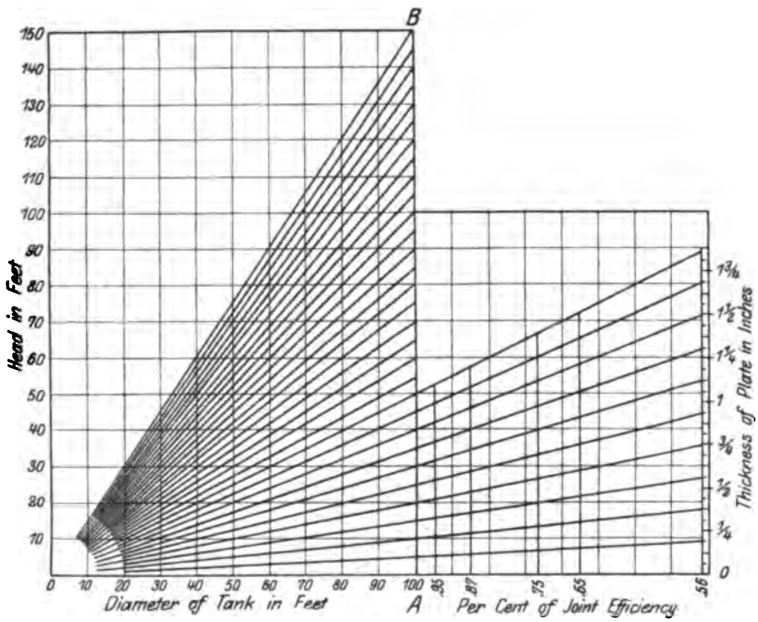


Fig. 45.

line, joint the point of intersection with origin. Where this line cuts the $A\ B$ axis, draw again a parallel to $0\ A$ until this meets a perpendicular line through the given diameter. Join this intersection with the origin 0 and continue until it meets the $A\ B$ axis, where allowable head is found.

45. Economical Diameter of Pressure Pipes.

In the following discussion let:

D = diameter of pipe in feet,
H = pressure head in feet,
e = riveted joint efficiency,
δ = safe tensile strength of metal per sq. in.,
t = thickness of shell,
W = weight in lbs. per lineal ft. of pipe,

$l =$ length of pipe line in feet,
$P =$ generated power,
$a =$ cost per horse power year at the station terminals,
$K =$ annual income,
$K' =$ annual net profit,
$F =$ fixed charges for the whole installation,
$F' =$ interest and depreciation of pressure pipe
$c =$ cost of steel per lb.,
$p =$ % depreciation and interest,
$Q =$ quantity of water in cu. ft. per sec.,
$h =$ loss of head,
$\varepsilon =$ efficiency of plant,
$S =$ slope of pipe line $= \frac{l}{h}$.

The thickness of pipe shell is determined by: $t = 2.6 \frac{HD}{e\delta}$.

If $e = 0.75$ and $\delta = 12500$, $t = \frac{HD}{3600}$.

The weight of a steel plate 12×12 and thickness t, weight of steel being taken as 490 lbs. per cu. ft. is $w = 41\,t$ lbs.

As the thickness increases in proportion to the head, and for the same head increases in proportion to the diameter, the average weight of one lineal ft. of pipe line is defined by $W = 41\,t \times \pi D$ or in function of the pressure head by $W = 0.036\,HD^2$.

If the weight of rivets, etc., is considered, 15% may be added and the expression becomes $W = 0.042\,HD^2$.

The total weight of the pipe line is denoted by

$$W' = 0.042\,l\,H\,D^2.$$

The annual income from sale of generated power is:

$$K = a\,P.$$

The interest and depreciation of pressure pipe has for value

$$F' = 0.042\,lcpHD^2.$$

The annual expenses are represented by

$$F + F' = F + 0.042\,lcpHD^2$$

The annual profits are given by the expression

$$K' = a\,P - (F + 0.042\,lcpHD^2) \quad \ldots\ldots\ldots \quad (1)$$

Power is denoted by

$$P = \frac{62.5}{550} Q\,(H - h)\,\varepsilon.$$

For $\varepsilon = 0.80$

$$P = \frac{Q\,(H-h)}{11} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

The Vallot formula for calculating pipes is:

$$D = 0.376 \left[\frac{Q}{\sqrt{S}}\right]^{\frac{2}{5}}$$

which is equivalent to this other

$$S = \sqrt[3]{\left[\frac{0.376 \sqrt[8]{Q^3}}{D}\right]^{16}}$$

$$S = 0.0054 \frac{Q^2}{D^{\frac{16}{3}}}$$

but $S = \frac{h}{l}$; therefore by substitution:

$$h = 0.0054 \frac{Q^2 l}{D^{\frac{16}{3}}}.$$

Placing this value of h in (2)

$$P = \frac{Q}{11}\left[H - 0.0054 \frac{Q^2 l}{D^{\frac{16}{3}}}\right].$$

The loss of income due to loss of head is:

$$F'' = 0.0005 \frac{Q^3 a l}{D^{\frac{16}{3}}}.$$

The annual net income will be

$$K = \frac{QaH}{11} - F - \left[F' + 0.0005 \frac{Q^3 a l}{D^{\frac{16}{3}}}\right] \quad \ldots\ldots\ldots \quad (3)$$

K will be a maximum when the third term of the second member is a minimum.

In order to simplify the calculations, let

$$u = 0.042\, lcpH.$$

$$v = 0.0005\, Q^3 a l.$$

By substitution, the third member of (3) becomes:

$$Z = uD^2 + \frac{v}{D^{\frac{16}{3}}}.$$

Differentiating and equating to zero:

$$\frac{dZ}{dD} = 2uD - \frac{16v}{2D^{\frac{19}{3}}} = 0.$$

Whence $\quad 3uD^{\frac{22}{3}} - 8v = 0$; and $D = \left[\frac{8v}{3u}\right]^{\frac{3}{22}} \quad \ldots\ldots\ldots \quad (4)$

Substituting the values of u and v, and reducing, the final expression for D is found to be

$$D = 0.625 \left[\frac{Q^3 a}{Hcp}\right]^{\frac{3}{22}} \quad \ldots\ldots\ldots\ldots \quad (5)$$

which is easily calculated by logarithms.

Example: Let

$Q = 165$ cubic feet per second,
$H = 1400$ feet,
$a = 40$ dollars,
$p = 0.10$ (0.05 interest and 0.05 depreciation),
$c = 0.05$ dollar.

Substituting in formula (5)

$$D = 0.625 \left[\frac{165^3 \times 40}{1400 \times 0.10 \times 0.05} \right]^{\frac{3}{22}} = 6.4 \text{ feet.}$$

The value of D so found corresponds to a maximum economy. In any case, it will be of value to first determine the diameter by this method, and then make such slight changes as local conditions will demand.

It will also be interesting to note that the figures applied in the above example are more or less the conditions that prevail at the Necaxa plant in Mexico, and the pressure pipes thereof are 6 feet in diameter.

46. Deformation of Pressure Pipes.

The hypothesis which always serves as a basis for the calculations to determine the thickness of steel pipe shells, the equality of pressure at all points of a transverse section of the pipe, is not sufficient in the design of penstocks of considerable proportions, and is strictly correct only in the case of a vertical pipe or one of small diameter.

The exact computations of pressures at every point of the shell, due to the weight of the water and the method of support of the pipe, offers much more complication. Steel pipes of circular cross-section, laid on the soil or on plane foundations, sustain elastic deformations which must be considered in the determination of shell thickness and reinforcement. A section primarily circular, takes an elliptical shape on account of the action of deforming forces, such as the weight of the shell considered by itself and the weight of water contained within.

The bending which results in these cases causes an additional and important stress, which is most evident in penstocks where the thickness of the shell is quite small in comparison with the diameter of the pipe. It has been found by analyzing a few examples corresponding to cases found in actual practice, that such stresses may be tenfold those produced solely by the pressure of water. Mr. Emil Kuichling has observed that empty steel pipes 3 to 6 ft. in diameter $\frac{1}{4}$ to $\frac{1}{2}$ in. thick, with 6 to 8 ft. of well compacted back fill, may show as much as 10% reduction in the vertical diameter, and states that ordinarily a depth of backfill of 5 to 6 ft. will produce stresses approaching the elastic limit.

The metal must therefore find itself subject to stresses that far exceed safe working amounts. Messrs. C. Birault and Forscheimer, in their studies of this subject, have considered independently the action of the weight of the shell alone and the action of the water contained within it.

They have demonstrated that at any point of the plate the elastic deformations due of the weight of the plate proper and the weight of the water are proportional.

Therefore, to ascertain the action of either force, it is necessary to multiply the results obtained with the other by a constant coefficient. The formula of M. Birault giving the bending moment at any point of the shell is, in case of a pipe lying on the bare soil

$$\frac{M}{Wr^2} = (\pi - \alpha) \sin \alpha - \frac{1}{2} \cos \alpha - 1 \quad \ldots \ldots (1)$$

where $M =$ bending moment at any point,
$W =$ weight of shell per foot length along the circumference,
$r =$ radius of the pipe,
$\alpha =$ angle AOB between the radius vector OB and radius through the origin OA, fig. 46.

The bending moment at C is given by the expression

$$m = -\frac{1}{2}Wr^2.$$

For $\alpha = 0$, it is found that

$$\frac{m'}{Wr^2} = -\frac{3}{2} \text{ or } m' = -\frac{3}{2}Wr^2 = 3m.$$

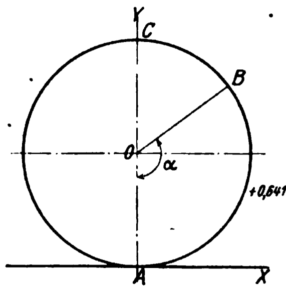


Fig. 46.

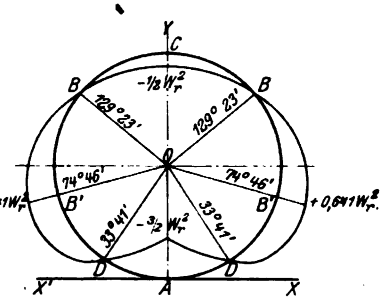


Fig. 47.

It is concluded then, that the bending moment at A is three times that at C and both being negative, the pipe is flattened at these places. To find the value of α which correspond to the maximum and minimum moments, the first differential of the second member of expression (1) is equated to zero and is solved for α, i. e.

$$\frac{1}{2}\tan\alpha = \pi - \alpha.$$

In one case $\alpha = \pi$ (upper section C), in the other $\alpha = 74^0\,46'$ (section $B'B'$).

The value of the maximum moment at B' fig. 47, is equal to

$$+0.641\,Wr^2$$

which being positive is inflated at this point. If the second member of (1) is equated to zero, the value for which the moments are zero will be found to be in one case:

$$\alpha = 33^0\,41' \ (DD)$$

in the other $\qquad \alpha = 129^0\,23' \ (BB)$

Plotting these and intermediate values as coordinates on the radii passing through these points, and scaling them off, starting from the circumference, due regard being paid to the sign, a curve shown in fig. 47 is obtained.

In the case of a pipe containing water, this water not being under pressure, the formula giving the bending moments due to the weight of the water only, is:

$$\frac{M}{\frac{1}{2}\gamma R^2} = (\pi - \alpha)\sin\alpha - \frac{1}{2}\cos\alpha - 1 \quad . \; . \; . \; . \; . \; (2)$$

It will be observed that the second member of this formula is equal, or rather identical, to the second member of expression (1). It is concluded, therefore, that the bending moments, and also the deformations, due to the weight of the shell and the water contained in the pipe, are truly proportional, and that the two effects can be combined.

If the water in the pipe is under pressure, this pressure is exerted normally on all elements of the shell, and along the circumference. In this case, the strains due to normal tensions alone need be considered, as the bending moments are equal to zero for any particular point.

The remedy for the deformations, pointed out above, is to reinforce the shell by means of circular bands or angles, the dimensions of which may easily be computed, after having determined the bending moment for any particular case under consideration. In all cases, it is more economical to adopt such rings, than to increase the thickness of the shell. That it is of the utmost importance to consider this subject in the design of steel pipes, is shown by the fact that the action due to weight of the water varies as the cube of the radius. The bending increases at a high rate with the increase of diameter, and the means of support, piers or other foundations, must be studied with care. It has been estimated by M. A. Bouchayer, that the flattening of a pipe attains its maximum before the pipe is full.

The above discussion, as has noted, covers the conditions of pipes laying on the bare soil. If, however, the pipe is buried under ground, the stresses due to the surrounding fill increase the complications of calculation, and can only be determined approximately. The factors that affect the pressure due to the fill, are several: the thoroughness of the tamping, the method of filling, and degree of humidity of the material.

If a pipe is buried under ground, the following conditions may develop:

Case No. 1: Concentrated loads at opposite points,
Case No. 2: Uniform loads in opposite directions,
Case No. 3: The pipe, either for case 1 or case 2, may be under pressure.

Case No. 1. Concentrated loads at opposite points, fig. 48.

Calling:

W = Concentrated load on unit length of pipe,
M_a = Bending moment at a,
M_b = Bending moment at b,
M_p = Bending moment at any point p,
φ = angle between the radius vector through the point p, and radius through the origin,
c = point of zero moment,
Θ = angle between radius vector through c, and radius through the origin,
c = unit compressive stress due to total thrust, c, being considered as uniformly distributed over the section,
t = thickness of the shell.

Then the bending moment at any point is given by

$$M_p = \frac{Wr}{2}[(1 - \cos\varphi) - (1 - \cos\Theta)] = \frac{Wr}{2}(\cos\Theta - \cos\varphi) \quad . \quad . \ (3)$$

The bending moment at a

$$M_a = \frac{W_r}{2}(0.6366 - 1) = -0.1817\, W_r \quad . \; . \; . \; . \; . \; (4)$$

The bending moment at b

$$M_b = \frac{W_r}{2}(0.6366 - 0) = +0.3183\, W_r \quad . \; . \; . \; . \; . \; (5)$$

(For a full discussion of this subject, see Bulletin No. 22 of the University of Illinois. Tests of Cast Iron and Reinforced Concrete Culvert Pipe. A. N. Talbot.)

It will be noted, that it is necessary to consider the compressive stresses in the shell due to direct thrust, in conjunction with the stresses due to the bending moment.

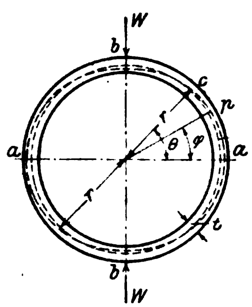


Fig. 48.

In fig. 48 the thrust at a due to the concentrated load W is $\frac{W}{2}$ and acts in a vertical direction; the thrust at b is zero. The vertical component of the direct thrust at any point p is $\frac{W}{2}$ and it is deduced that

$$C = \frac{W}{2}\cos\varphi$$

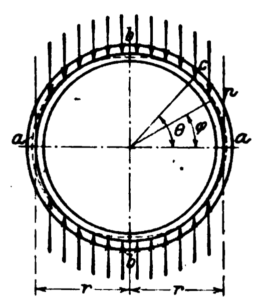


Fig. 49.

from which

$$c = \frac{W}{2\,t}\cos\varphi \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (6)$$

Case 2: Uniform loads in opposite directions, fig. 49.

All symbols as for case 1, ω = load per unit width of projection for unit length of pipe.

The bending moment at any point p is obtained by taking moments at p, whereby

$$M_p = \frac{\omega r^2}{2}\sin^2\varphi - M_a = M_b - \frac{\omega r^2}{2}\cos^2\varphi \quad . \; . \; . \; . \; (7)$$

But it is known that

$$M_a = -M_b\,.$$

It is found therefore:

$$M_a = -\frac{1}{4}\,\omega r^2 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (8)$$

$$M_b = +\frac{1}{4}\,\omega r^2 \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (9)$$

If it be required to find the point for which the moment is zero, M_p must be equated to zero in equation (7)

$$M_p = 0 = M_b - \frac{\omega r^2}{2}\cos^2\varphi = \frac{1}{4}\,\omega r^2 - \frac{\omega r^2}{2}\cos^2\varphi\,.$$

From which is deduced

$$\cos^2\varphi = \frac{1}{2}; \quad \text{or} \quad \varphi = 45^0\,.$$

The bending moment at any point p may also be calculated in function of the vertical and horizontal components of the acting forces at that point, from the following relation:

$$M_p = \left(1 - \frac{\omega_h}{\omega_v}\right) \cdot \left(M_b - \frac{\omega r^2}{2} \cos^2 \varphi\right) \quad . \; . \; . \; . \; . \; (10)$$

in which $\omega_v =$ Intensity of the vertical forces
$\omega_h =$ Intensity of the horizontal forces.

In this last equation, it will be noted that the bending moment at any point is zero, if the components have the same value.

In fig. 49, the direct thrust at a is ωr, and the direct thrust at b is zero.

At any point p the vertical component of the direct thrust is

$$\omega r \cos \varphi$$

and the tangential thrust is

$$C = \omega r \cos^2 \varphi$$

wherefrom

$$c = \frac{\omega r}{t} \cos^2 \varphi \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; (11)$$

Therefore, to determine the thrust when the pipe is subjected to both horizontal and vertical loads, uniformly distributed over the projections of the pipe, formula (11) must to be used.

Case 3. The pipe is under pressure.

Make all calculations for concentrated loads at opposite points, or for uniform loads in opposite directions, as the case may be, and separately compute the stresses due to pressure, finally combining the two effects, with due regard to the directions of the acting forces.

47. Specifications for Steel Pipes.

Material to be medium steel. Quality of steel and workmanship of same to be according to Manufacturers' Standard Specifications for such material. All inside edges to be bevel-sheared for caulking.

Rivet holes must be accurately aligned and matched, and shall be either drilled or reamed parallel after punching.

Entire work to be shipped knocked-down, with all parts clearly marked with a steel stencil in conformity with an erection drawing to be furnished by the manufacturer.

Rivets to be furnished throughout and include 5% excess for waste. Fitting-up bolts are to be furnished, the number being 10% of rivets. The entire work to be thoroughly cleaned of all rust and scale and then coated with pure linseed oil before shipment. Detailed shop drawings to be submitted to the engineers (purchasers) for their approval before work is commenced.

Entire work as to quality of material, workmanship, etc., must be to the satisfaction of the purchasers.

The purchasers reserve the right to inspect the work before shipment; such inspection, however, shall be considered only as an aid to the manufacturer, and shall not relieve the manufacturer from any or all responsibility, regarding defective material or workmanship, or regarding inaccuracies in the work, which may appear when the whole is assembled and erected at the purchasers works.

48. Pipe Coating.

Protection of steel pipe is the most important factor in its lasting quality.

There are two methods of protecting pipes: one is the metallurgical process, consisting in alloying the iron with another metal, securing a product which will not corrode under exposure; the other process is mechanical in character, and consists in covering the surface of the metal with some substance which will secure it against chemical or physical change under exposure. The second method has the advantage of being the cheaper one.

A preservative used with much success is asphaltum, made from gilsonite, practically a pure hydro-carbon. This material is of such physical properties that it does not become brittle, crack, nor melt in warm weather. The pipe is submerged in that bath, which is kept at a temperature of 400° Fahr., then drained, leaving an even and protective coating, inside as well as outside.

Another pipe coating, very efficient also, can be prepared from coal tar pitch distilled until the naphtha is entirely removed, then being mixed with from 5% to 6% of linseed oil, heated to 300° Fahr.; this temperature being maintained throughout the operation of dipping.

The specifications for the coating of the 24 mile steel pipe line for the city of Portland, Ore., were as follows: "After cleaning and inspection, which must be done under cover, every pipe shall be coated with "Mineral Rubber Coating", or such other material equal thereto in all its qualities, as hereinafter specified.

The coating must be free from blisters and bubbles, and must not be affected by exposure nor by the action of soil or salt water. It must strongly adhere to the pipe under all circumstances, and must not become soft enough, to flow at a temperature of 150° Fahr., nor brittle enough to crack or scale off at freezing temperature.

The pipes before being dipped must be clean and free from rust, and no pipe shall be coated unless examined and approved by the inspector immediately before the process of coating begins. They shall be dipped in a bath of the "Mineral Rubber Coating" heated to 400° Fahr., or more, and shall receive a uniform coating of not less than $\frac{1}{16}$ inch in thickness. While in the bath, the pipe shall be so manipulated, scraped, brushed and re-dipped, if necessary, in accordance with the directions of the inspector, as to give a sound and perfect coating. After the pipes have been removed from the dipping tank, they shall be set vertically while cooling. Proper facilities for handling the pipes, and allowing all surplus material to drip off shall be provided by the contractor. The cost of all labor and material involved in the coating of the pipes must be included in the price bid for furnishing and laying said pipes.

The materials for the mixture, the appliances and method of boiling, melting, or applying and testing the coating, shall be subject to the approval of the engineer or his authorized inspector. In case that from any cause the coating on any length may be found to be defective, it must be removed, and should the defect be from brittleness or scaling, or improper composition or application of the coating, it must be scraped off entirely, outside and inside, and another coating applied."

49. Expansion Joints.

Owing to variations in temperature, steel pipes undergo a large amount of expansion and contraction. To take care of this, it is customary to use expansion joints of the types shown in the figures 50—54, and placed at intervals of 500 feet, more or less.

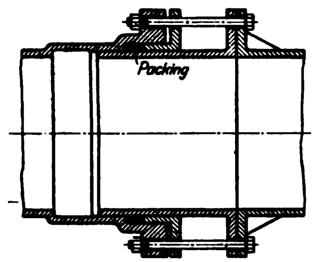


Fig. 50.

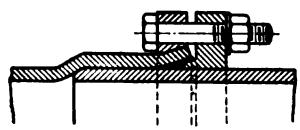


Fig. 53.

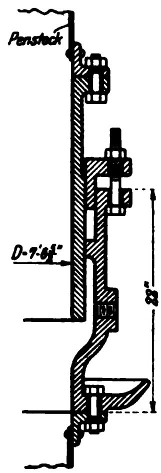


Fig. 51.

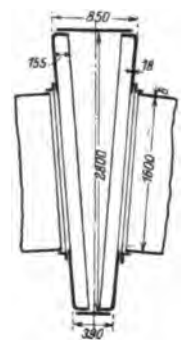


Fig. 52.

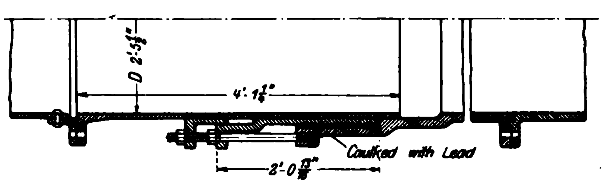


Fig. 54.
Typical Expansion Joints.

The best location for expansion joints is near the elbows encountered in the pipe line. This joint takes care of expansion and contraction in itself, and being slightly flexible although absolutely tight under pressure, it allows the pipe to conform with the grade, and permits a slight deflection without leakage.

50. Manholes.

Manholes should be distributed along a pipe line and distanced apart as local conditions may require. They should be carefully referenced. Such manholes have the cover bolted on, which, when the pipe is empty and necessary examinations are to be made, may be removed to allow a man to enter. Fig. 55.

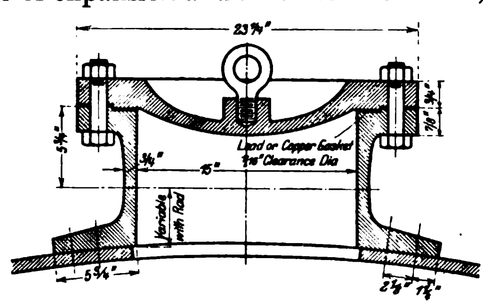


Fig. 55.
Details of Manhole and Cover.

The cover and its attachment are designed so as to withstand the static pressure and water hammer effect, and the diameter must be large enough to allow a man to enter without difficulty.

The static pressure acting on such a cover is easy to determine: if

$p =$ pressure to be determined, in lbs.
$H =$ static head, in feet
$A =$ area of cover, in sq. ft.
$= \pi r^2$, if circular, $= \pi a b$, if elliptical,

then, the required pressure will be found by the expression:

$$p = 62.5\ A\,H\,.$$

If a water hammer effect is possible, its magnitude is to be calculated separately and the result added to the static effect. The number and size of the bolts are easily established therefrom.

51. Area of Gate Opening.

When a circular gate is lifted to a certain height, the clear opening takes the shape of a half moon, the area of which may have to be determined.

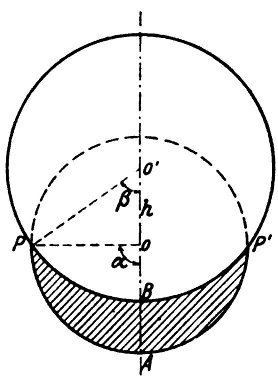


Fig. 56.

Call:

$r = OA$ the radius of the pipe,
$R = O'B$ the radius of the gate,
$h = OO'$ the distance between centers,
$\alpha =$ angle POA,
$\beta =$ angle $PO'A$,
$S =$ area of clear opening.

The problem is to caleulate S., being given r, R and h; α and being β auxiliary variables.

From figure 56 we have

area $PBA =$ area $POA +$ area POO' -- area $PO'B$.

In other words

$$\frac{S}{2} = \frac{r^2}{2}\alpha + \frac{rh}{2}\sin\alpha - \frac{R^2}{2}\beta$$

or $S = r^2\alpha + rh\sin\alpha - R^2\beta$ . . . . (a)

From the triangle POO', we have

$$\frac{r}{\sin\beta} = \frac{R}{\sin\alpha} \quad \text{. . . . . (b)}$$

$$R^2 = h^2 + r^2 + 2hr\cos\alpha \quad \text{. . . . . (c)}$$

Therefore, to determine S, the unknown of the problem, α is obtained in function of h in equation (c), β is obtained in formula (b), these two values so found and substituted in (a) give S.

Table XXVIII gives the ratio of S to A (the area of the pipe section) and these figures are sufficiently exact for practical purposes.

Numerical example: A gate valve 36 inches in diameter, is completely opened in 42 turns.

If 12 turns only are given, determine the area of clear opering.

First we find the ratio of turns given to the total number of turns possible

$$\frac{12}{42} = \frac{2.85}{10}\,.$$

By interpolation is found

$$\frac{S}{A} = 0.2 + 0.128 \times 0.85 = 0.309.$$

Therefore
$$S = 3.1416 \times \overline{1.5}^2 \times 0.309 = 2.18 \text{ sq. ft.}$$

which is
$$\frac{2.18 \times 100}{3.1416 \times \overline{1.5}^2} = 30.8\,\%$$

approximately of the total opening.

Table XXVIII.

F	$\frac{S}{A}$	F	$\frac{S}{A}$
$\frac{1}{10}$	0.100	$\frac{6}{10}$	0.705
$\frac{2}{10}$	0.228	$\frac{7}{10}$	0.805
$\frac{3}{10}$	0.356	$\frac{8}{10}$	0.891
$\frac{4}{10}$	0.477	$\frac{9}{10}$	0.962
$\frac{5}{10}$	0.595		

F = fraction of opening.

52. The Elbow.

A problem which is often present in the design of pipe lines is that of the elbow; a careful study must always be made of the forces that tend to pull an elbow from its anchorage, thus producing dangerous strains in the pipe itself if not properly secured. In a static condition, when water is not flowing, the faces of the elbow, fig. 57 AB and CD receive normal pressures, which are transmitted integrally to the section DB, so to compute the pressure which is exerting itself on the shell at that point, it is necessary to find the resultant of the two normal pressures.

These pressures are each equal to

$$P = 62.5\,AH$$

where A = is the area of the section of pipe,
and H = is the static head.

The resultant is therefore:

$$R = 125\,AH \cos\frac{x}{2}.$$

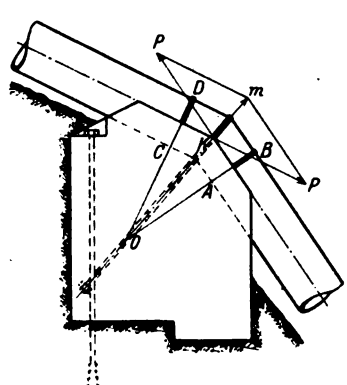


Fig. 57.

This relation is immediately observed in fig. 57 where the isosceles triangle KPm gives:

$$Km = R = 2\,KP \cos PKm.$$

There is also the centrifugal force to be added when the water is flowing; this centrifugal force has for expression:

$$F = \frac{M v^2}{R}$$

M being the mass of moving water,
v its velocity, and
R radius of curvature of the elbow.

To calculate M, call W the weight of water contained between the faces AB and CD and find:
$$M = \frac{W}{g} = \frac{W}{32.16}$$

The elbow must therefore be solidly bolted to a pier which will offer resistance by its own weight. The resultant of the masonry and water forces should intersect the base of the pier within the middle third to provide a safe factor against overturning. In special cases of sharp turns, the pier itself must be strongly anchored, as in fig. 57.

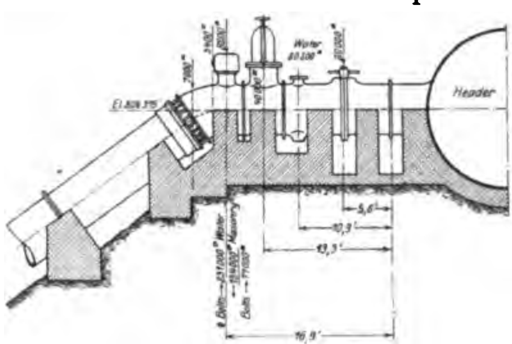


Fig. 58.
Calculations for Elbow Anchorage.
Great Western Power Co.'s Pipe Lines.

Fig. 58 shows the conditions that were met with in the case of the elbows of the Great Western Power Co.'s pressure pipes. These elbows are connecting the pressure pipes to the header.

The bases for calculations were as follows:

Elevation of high water		1025
Elevation of center line of penstock		806
	Effective head	219 feet
Rise in dynamic head due to closing of gates		114 ft.
Total head considered		333 ft.

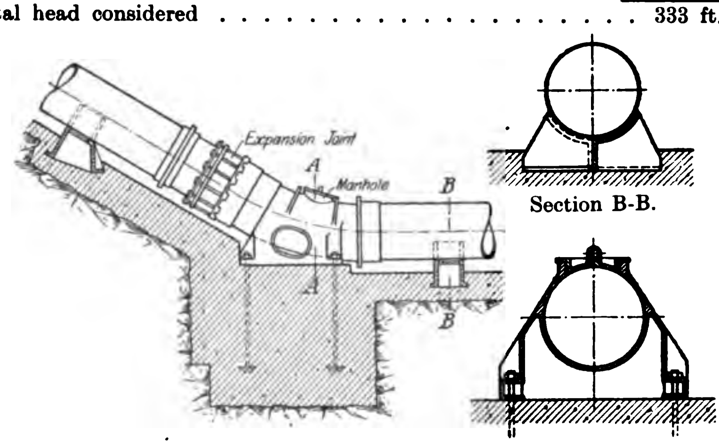


Fig. 59. Section A-A.
Elbow with Manhole and Expansion Joint. German Design.

The weights were taken as follows (see fig. 58):

Butterfly valve	20,000 lbs.
Gate valve	40,000 „
Elbow and air valve	13,600 „
Expansion joint	7,000 „

Considering the weight of the water as far as point *B*, the weight of masonry as 140 lbs. a cubic foot, and the whole metallic system being anchored into it, it was found that 77,000 lbs. had to be taken care of by bolts fastened into the rock.

Fig. 59 and 60 show foreign practice in elbow anchorage.

53. Blow-offs.

These are located at each depression of the pipe line where sediment may accumulate and thereby affect the carrying capacity of the pipe.

Air valves: They are generally called relief valves and are designed to operate automatically, being located at the summits of a pipe line. They permit the escape of air in filling, and the entrance of air on emptying a pressure pipe. Such inlet valves are illustrated in fig. 61 and 62 and should be designed to admit the necessary volume of air without a greater pressure difference, across the valve, than 3 lbs. per sq. inch.

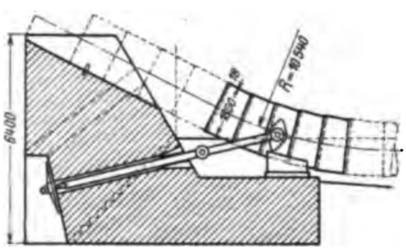


Fig. 60.
Elbow Anchorage, German Design.

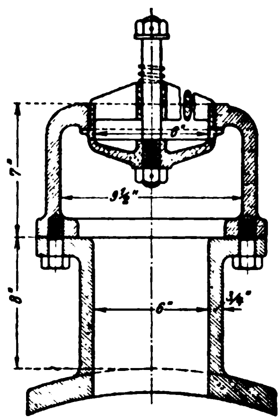


Fig. 61.
Details of 6″ Air Valve.

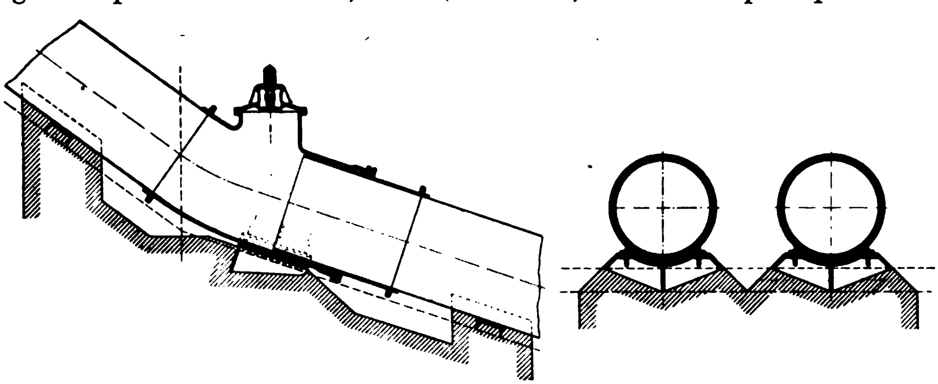


Fig. 62.
Air Valve and Expansion Joint in Penstock Run. Swiss Design.

A collapse of 7½ sections of 44 in. pipe, ¼″ thick, of the Bull Run pipe line occurred from external air-pressure while under test. The water had been running two days when the collapse occurred; no air valves had been placed.

54. Piers.

If the pipes are supported by foundations or piers, and these are shaped to receive the circular form of the penstocks, it will be noted by referring to fig. 63 that the part under the most important strain is relieved from fatigue. The supports being raised up to a point as *B* for example, the resistance of the pier at such a point will oppose itself to the deforma-

tion. The bending moments shown in fig. 47 are positive outside and negative inside of the circle. When the pipes are laid and supported by piers, the allowable distance from pier to pier must be carefully determined, the maximum bending moment and shear at the points of support being found by comparing the pipe to a continuous beam supporting a uniform load equal to its own weight and that of the water inside. The piers are usually to be provided with plates extending along nearly the lower half of the circumference of the pipe. In this way, they offer less resistance to the movements of the pipe due to expansion.

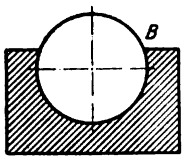


Fig. 63. Typical Pier for Penstock.

55. The Siphon.

The determination of the profile of a pipe line is generally governed by the location of the intake and the power house or surge chamber as the case may be. When the line is to be laid over ground which is higher than the hydraulic gradient, a siphon is commonly used. The flow of water in a siphon is calculated from the relation $h =$ loss at entrance $+$ friction loss $+$ loss at exit, or in mathematical form:

$$h = \frac{v^2}{2g} + lS + \frac{v_1^2}{2g} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

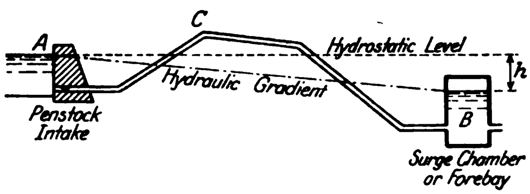


Fig. 64a. Pipe Line above Hydrostatic Level.

It is well to note that the height of the siphon above hydrostatic level (fig. 64a) should not exceed 34 feet theoretically (at sea level). In practice it should be less than the barometric height by a amount

$$h = \frac{v^2}{2g} + l_1 S + \frac{v_1^2}{2g}$$

where l_1 is the length of the pipe line from A to C or to the highest point of the siphon.

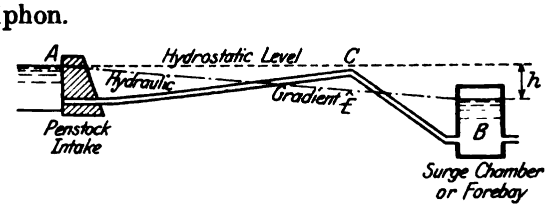


Fig. 64b. Pipe Line above Hydraulic Gradient.

At the summit, it is common practice to locate an air valve because the air accumulates there during the working of the pipe system; on the other hand, a vacuum could be produced on closing the penstock intake, in which case, the pipe may collapse, unless provision is made for the admission of air.

Naturally, to start the flow in the pipe, it will be necessary to fill it by means of a pumps or other system. If a pipe line is to be laid over ground as per figure 64b, the pressure at C will be less than the barometric pressure by a head equal to CE. The discharge will not be altered for a given length of pipe line. In this case, it is not necessary to "charge" the siphon, because C is below the hydrostatic level. However, air troubles must be expected unless air valves are provided.

A case frequently met with is that of the reversed or inverted siphon, this being a practical method to cross valleys. In fig. 64a, 64b, 65, A is the penstock intake and B is a chamber or stand pipe to receive the effects of the swelling harmonic waves produced by the sudden shut-down at the power house.

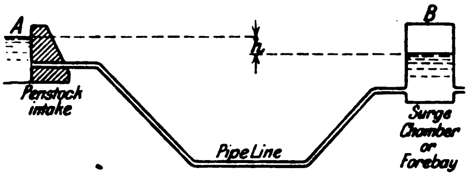


Fig. 65. Inverted Siphon.

In calculating any of the three cases, the fundamental equation (1), as given above, is to be used.

Table XXIX. *Atmospheric pressure at different elevations.*

Elev. above sea level. Feet	Press. in lbs. per square Inch.	Height of Mercury Barometer. Inches.	Height of Water Barometer. Feet
0	14.7	30.00	34.0
500	14.5	29.47	33.3
1000	14.2	28.94	32.8
2000	13.7	27.92	31.6
4000	12.7	25.98	29.4
6000	11.8	24.18	27.4
8000	11.0	22.50	25.5
10000	10.3	20.93	23.7

Table XXIX may be of use in obtaining the height of the mercury barometer corresponding to elevations above sea level up to 10000 feet. The last column gives the height of the water barometer and is useful for determining whether excavation is nesessary to keep the pipe line within the allowable limit above the hydraulic gradient.

56. Protection against Water freezing inside of Steel Pipes.

The pipes being exposed to climatic changes, it is necessary to prevent the effects of the heat, and the possibility of freezing. In the first case, an easy remedy is to cover the shell with white paint.

As the velocity of the water in the pipes is at least 3 ft. per second to prevent deposits of sand etc., the water can not easily freeze unless the temperature is very low. But in case of very cold weather, while the plant is shut down for one reason or another, should the flow of water be stopped, this would freeze and cause damage. It is necessary, then, to let the water flow through a by-pass, or other means, at such a velocity as will prevent freezing.

To determine what number of heat units per hour will be produced and lost to the air, the following formula can be applied:

$$H_u = kD\pi l\,(t_a - t_w)$$

in which

k = a constant coefficient, taken as 4.55 usually,
t_w = mean temperature of the water,
t_a = temperature of the atmosphere,
D = outside diameter of the pipe,
l = length of the pipe exposed to the air.

One cubic foot of water in cooling one degree, gives off

$$62.5\ B.T.U.$$

The quantity of water passing through the pipe in one hour is

$$Q = \frac{H_u}{62.5\,(t_w - \frac{1}{2})}$$

when the water is allowed to cool one half degree only.

Calling $A = D\pi l$ the exposed surface of the pipe, and if $(t_a - t_w)$ is taken as 36 degrees, the following formula is deduced:

$$Q = \frac{4.55 \times A \times 36}{31.25} = 5.05\,S$$

approximately.

Practically $Q = 5S$.

For instance, assume a pipe of 4 feet diameter, 1500 ft. long. The area exposed to the atmosphere is

$$A = D\pi l = 4 \times 3.14 \times 1500 = 18{,}850 \text{ square feet.}$$

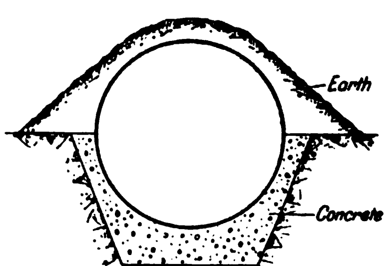


Fig. 66.

Therefore

$$Q = 94{,}250 \text{ cu. ft. per hour or}$$
$$= 26.1 \text{ cu. ft. per second.}$$

The velocity that will prevent freezing is therefore

2.07 feet per second.

Wherever possible in the case of very cold climates, the pipe should be buried into the ground, fig. 66.

57. Wind Pressure on Circular Conduits.

In the design of penstock bridges, it is necessary to compute the wind stresses due to pressure on the supported pipe. Mr. T. M. Gilmer gives the following method in the *Engineering Record.*

Let the unit pressure be p.

Then the pressure on any small element $hrd\alpha$ will be

$$(hrd\alpha)\,p\sin^2\alpha.$$

The component of the normal pressure parallel to $0x$, fig 67, is

$$hrd\alpha\,p\sin^3\alpha.$$

The total pressure in the direction $0x$ is

$$P = 2\,hrp \int_0^{\frac{\pi}{2}} \sin^3 \alpha\, d\alpha = \frac{4}{3}\, hrp\,.$$

This is the pressure on the cylindrical surface in the direction of OX in terms of the radius and the unit pressure. Its relation to the pressure on a plane surface of the same length and of a width equal to the diameter

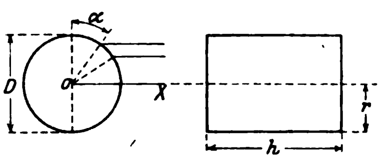


Fig. 67.

of the pipe may be found as follows: The total pressure on a plane surface, P, equal to the diametral section, is:

$$P' = 2\,hrp,$$

therefore

$$P = \frac{2}{3}\, P'$$

or the pressure on the cylindrical surface is two thirds that on its diametral projection.

58. Flanges.

Pressure pipe flanges must be attached securely and rigidly to the pipe, making a tight joint and eliminating absolutely the possibility of a

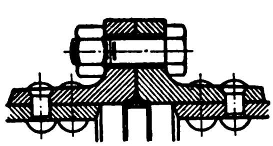


Fig. 68.

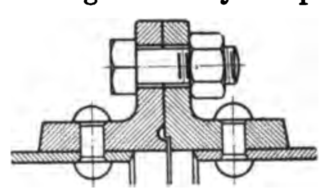


Fig. 69.

leak. Flanges for pressure pipes are preferably made of forged steel; and this insures their safety in transit and in installation. It is often desirable that a joint be slightly flexible, and although, as such, it will allow the pipe

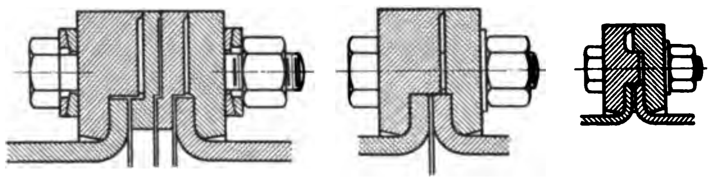


Fig. 70.
Details of Flanges of the Necaxa Penstocks. Mexico.

to conform with grade, it must be absolutely tight under pressure. The design of flanges is a very important part in the study of pressure pipe lines, and the few accompanying types, figs. 68, 69, 70, will, no doubt, be of valuable interest to the designing engineer.

59. Riveted Joints.

The following formula is to be used in determining the pitch of rivets:

$$p = \frac{s \cdot A}{t \cdot Q} + d$$

where

p = pitch of rivets,
A = area of rivet hole in decimal of an inch,
s = shearing value of rivet,
t = thickness of plate,
Q = tensile strength of plate,
d = diameter of rivet hole in inches.

The thickness of metal to be used for a pressure pipe may be determined as follows:

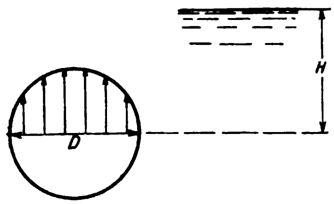


Fig. 71.

Let

H = head at any point in feet,
D = diameter of pipe in feet.

The pressure upon one half the shell will be (Fig. 71):

$$\frac{D \cdot H \cdot 62.5}{2}.$$

The resisting effort of the pipe is offered by the thickness of the plate, and the riveting, such an effort depending upon:

δ = 50,000 lbs per sq. in. = tensile strength of material,
e = coefficient of joint efficiency,
F = factor of safety,
l = unit length of pipe = 12 inches.

The resisting effort is then given by:

$$\frac{\delta \cdot e \cdot l}{F}$$

and the thickness of plate is:

$$t = \frac{\dfrac{D \cdot H \cdot 62.5}{2}}{\dfrac{\delta \cdot e \cdot l}{F}} = \frac{F \cdot D \cdot H \cdot 31.25}{\delta \cdot e \cdot l}.$$

For $\delta = 50{,}000$ lbs.; $F = 4$, $l = 12$

this formula becomes

$$t = \frac{D \cdot H}{4800 \cdot e} \quad \ldots \ldots \ldots \ldots \quad (1)$$

The relation of size of rivet to the thickness of plate is established in Table XXX.[1]

In the following table, the length includes the length of shank necessary to form the field head, and for a grip equal to twice the thickness of plate assumed.

[1] Bureau of Construction and Repair. Navy Department.

Table XXX.

Thickness of Plate. Inches	Diameter of Rivet. Inch	Diameter of Rivet. Decimal	Corresponding Rivet Hole Area. Inch	Corresponding Rivet Hole Area. Decimal		Length. Inches
Less than $\frac{3}{16}$ . . .	$\frac{3}{8}$	0.3750	$\frac{7}{16}$	0.4375	0.1503	$\frac{7}{8}$
$\frac{3}{16}$ to $\frac{1}{4}$ . . .	$\frac{1}{2}$	0.5000	$\frac{9}{16}$	0.5625	0.2485	1
$\frac{1}{4}$ to $\frac{5}{16}$ . . .	$\frac{5}{8}$	0.6250	$\frac{11}{16}$	0.6875	0.3712	$1\frac{1}{4}$
$\frac{5}{16}$ to $\frac{1}{2}$ . . .	$\frac{3}{4}$	0.7500	$\frac{13}{16}$	0.8125	0.5185	$1\frac{3}{4}$
$\frac{1}{2}$ to $\frac{3}{4}$ . . .	$\frac{7}{8}$	0.8750	$\frac{15}{16}$	0.9375	0.6903	$2\frac{1}{4}$
$\frac{3}{4}$ to 1 . . .	1	1.0000	$1\frac{1}{16}$	1.0625	1.0031	$2\frac{7}{8}$

The centers of rivets are spaced not less than $1\frac{5}{8}$ times their diameter from the edges. In double and treble riveting, their distance from center to center of rows (horizontal pitch) to be not less than $2\frac{1}{4}$ diameters in laps, and $2\frac{1}{2}$ diameters for straps.

It is noted here that transversal riveting should be only one half of longitudinal riveting.

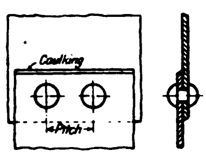


Fig. 72.

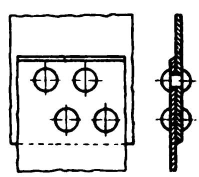


Fig. 73.

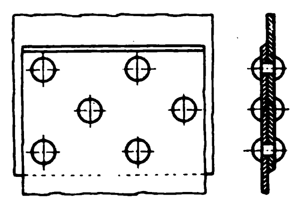


Fig. 74.

There are several kinds of riveted joints:

single riveted joint (fig. 72) if only one row of rivets is used;
double riveted joint (fig. 73) if two rows of rivets are used;
triple riveted joints and quadruple riveted if three and four rows are used respectively (fig. 74).

A riveted double welt butt-joint consists of one or more rows of rivets to join two plates, covered on both in- and out-side with covering strips (fig. 75).

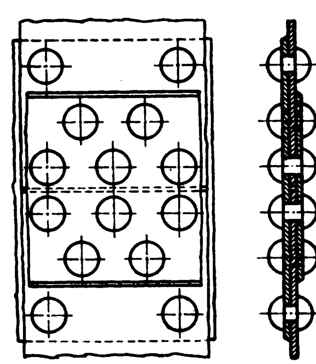


Fig. 75.

It is obvious that in increasing the efficiency of the joints, by adopting several rows of rivets, which is imperative for very high heads, a loss of head is produced which may reach important proportions. It is the practice to use seamless welded pipes in such instances, and this use is becoming general.

Table XXXI[1] of maximum efficiency of riveted joints and the corresponding pitch of rivets for wrought iron and steel plates, will be of much interest to the designers of pressure pipes.

[1] Engineering News Vol. L. No. 3. — Page 68.

Table XXXI. *Table of maximum efficiency of riveted joints and the corresponding pitch of rivets for wrought iron and steel plates.*

Thickness of Plate	$\frac{3}{16}=.1875$				$\frac{1}{4}=.25$					
Diameter of Rivets	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$
Diam. of Rivet Hole	$\frac{7}{16}$	$\frac{8}{16}$	$\frac{9}{16}$	$\frac{10}{16}$	$\frac{8}{16}$	$\frac{9}{16}$	$\frac{10}{16}$	$\frac{11}{16}$	$\frac{12}{16}$	$\frac{13}{16}$
Area of Rivet Hole	.150	.196	2.49	.307	.196	.249	.307	.371	.442	.518
Area $\times$ 38,000 . . .	5,700	7,448	9,467	11,670	7,448	9,462	11,670	14,100	16,800	19,680
P. Single Riv. Pitch	$\frac{15}{16}$	$1\frac{3}{16}$	$1\frac{7}{16}$	$1\frac{11}{16}$	1	$1\frac{3}{16}$	$1\frac{3}{8}$	$1\frac{5}{8}$	$1\frac{7}{8}$	$2\frac{1}{8}$
Efficiency of Joint	53	58	61	63	50	53	55	58	60	62
P. Double Riv. Pitch	$1\frac{7}{16}$	$1\frac{13}{16}$	$2\frac{1}{4}$	$2\frac{11}{16}$	$1\frac{1}{2}$	$1\frac{13}{16}$	$2\frac{3}{16}$	$2\frac{9}{16}$	3	$3\frac{7}{16}$
Efficiency of Joint	70	72	75	77	67	69	71	73	75	76
P. Triple Riv. Pitch	$1\frac{15}{16}$	$2\frac{1}{2}$	$3\frac{1}{16}$	$3\frac{3}{4}$	2	$2\frac{7}{16}$	$2\frac{15}{16}$	$3\frac{1}{2}$	$4\frac{1}{8}$	$4\frac{3}{4}$
Efficiency of Joint	78	80	82	83	75	77	79	80	82	83

Thickness of Plate	$\frac{5}{16}=.3125$					$\frac{3}{8}=.375$				$\frac{7}{16}=.4375$			
Diameter of Rivets	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$
Diam. of Rivet Hole	$\frac{9}{16}$	$\frac{10}{16}$	$\frac{11}{16}$	$\frac{12}{16}$	$\frac{13}{16}$	$\frac{12}{16}$	$\frac{13}{16}$	$\frac{14}{16}$	$\frac{15}{16}$	$\frac{13}{16}$	$\frac{14}{16}$	$\frac{15}{16}$	$\frac{16}{16}$
Area of Rivet Hole	.249	.307	.371	.442	.518	.442	.518	.601	.690	.518	.601	.690	.785
Area $\times$ 38,000 . . .	9,462	11,670	14,100	16,800	19,680	16,800	19,680	22,840	26,220	19,680	22,840	26,220	29,830
P. Single Riv. Pitch	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{7}{16}$	$1\frac{5}{8}$	$1\frac{7}{8}$	$1\frac{1}{2}$	$1\frac{11}{16}$	$1\frac{7}{8}$	$2\frac{1}{8}$	$1\frac{9}{16}$	$1\frac{3}{4}$	$1\frac{15}{16}$	$2\frac{1}{8}$
Efficiency of Joint	47	50	52	54	57	50	52	53	56	48	51	52	53
P. Double Riv. Pitch	$1\frac{9}{16}$	$1\frac{7}{8}$	$2\frac{3}{16}$	$2\frac{9}{16}$	$2\frac{15}{16}$	$2\frac{1}{4}$	$2\frac{9}{16}$	$2\frac{7}{8}$	$3\frac{1}{4}$	$2\frac{5}{16}$	$2\frac{5}{8}$	$2\frac{15}{16}$	$3\frac{1}{4}$
Efficiency of Joint	64	67	69	71	72	67	68	69	71	65	67	68	69
P. Triple Riv. Pitch	$2\frac{1}{16}$	$2\frac{1}{2}$	$2\frac{15}{16}$	$3\frac{7}{16}$	$3\frac{15}{16}$	6	$3\frac{7}{16}$	$3\frac{15}{16}$	$4\frac{7}{16}$	$3\frac{1}{16}$	$3\frac{1}{2}$	$3\frac{15}{16}$	$4\frac{3}{8}$
Efficiency of Joint	73	75	77	78	79	75	76	77	79	74	75	76	77

Thickness of Plate	$\frac{1}{2}=.5$				$\frac{9}{16}=.5625$				
Diameter of Rivets	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$
Diam. of Rivet Hole	$\frac{13}{16}$	$\frac{14}{16}$	$\frac{15}{16}$	$\frac{16}{16}$	$\frac{14}{16}$	$\frac{15}{16}$	$\frac{16}{16}$	$1\frac{1}{16}$	$1\frac{2}{16}$
Area of Rivet Hole	.518	.601	.690	.785	.601	.690	.785	.887	.994
Area $\times$ 38,000 . . .	19,680	22,840	26,220	29,830	22,840	26,220	29,830	33,710	37,770
P. Single Riv. Pitch	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{13}{16}$	2	$1\frac{9}{26}$	$1\frac{11}{16}$	$1\frac{7}{8}$	$2\frac{1}{16}$	$2\frac{1}{4}$
Efficiency of Joint	46	46	48	50	44	44	47	48	50
P. Double Riv. Pitch	$2\frac{1}{8}$	$2\frac{3}{8}$	$2\frac{11}{16}$	3	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$	$3\frac{1}{16}$	$3\frac{3}{8}$
Efficiency of Joint	62	63	65	67	61	63	64	65	67
P. Triple Riv. Pitch	$2\frac{13}{16}$	$3\frac{3}{16}$	$3\frac{9}{16}$	4	$2\frac{15}{16}$	$3\frac{1}{4}$	$3\frac{5}{8}$	$4\frac{1}{16}$	$4\frac{1}{2}$
Efficiency of Joint	71	72	74	75	70	71	72	74	75

Thickness of Plate	$\frac{5}{8}=.625$					$\frac{3}{4}=.75$			
Diameter of Rivets	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$
Diam. of Rivet Hole	$\frac{15}{16}$	$\frac{16}{16}$	$1\frac{1}{16}$	$1\frac{2}{16}$	$1\frac{3}{16}$	$\frac{16}{16}$	$1\frac{1}{16}$	$1\frac{2}{16}$	$1\frac{3}{16}$
Area of Rivet Hole	.690	.785	.887	.994	1.110	.785	.887	.994	1.110
Area $\times$ 38,000 . . .	26,220	29,830	33,710	37,770	42,180	29,830	33,710	37,770	42,180
P. Single Riv. Pitch	$1\frac{5}{8}$	$1\frac{13}{16}$	$2\frac{15}{16}$	$2\frac{5}{8}$	$2\frac{15}{16}$	$1\frac{11}{16}$	$1\frac{13}{16}$	$1\frac{15}{16}$	$2\frac{1}{8}$
Efficiency of Joint	42	44	45	47	49	41	41	42	44
P. Double Riv. Pitch	$2\frac{5}{16}$	$2\frac{9}{16}$	$2\frac{7}{8}$	$3\frac{1}{8}$	$3\frac{7}{16}$	$2\frac{5}{16}$	$2\frac{9}{16}$	$2\frac{13}{16}$	$3\frac{1}{16}$
Efficiency of Joint	59	61	63	64	65	57	58	60	61
P. Triple Riv. Pitch	$3\frac{1}{16}$	$3\frac{3}{8}$	$3\frac{3}{4}$	$4\frac{1}{8}$	$4\frac{3}{16}$	3	$3\frac{5}{16}$	$3\frac{5}{8}$	4
Efficiency of Joint	69	70	72	73	74	67	68	69	70

Thickness of Plate	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$
A = Distance of C. L. of riv. to edge of plate .	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2	$2\frac{1}{8}$

B = Transverse pitch of rivets for double and triple riveted joints = $\frac{3}{4}$ of above pitches to nearest $\frac{1}{16}$ inch.

Pitches calculated so that tensile strength of plate at 60,000 lbs per sq. in. equals shearing strength of rivets at 38,000 lbs per sq. in.; Diam. of rivet holes being used to obtain rivet area.

$$\text{Efficiency} = \frac{\text{net section plate}}{\text{gross section plate}}.$$

60. Water Hammer in Pressure Pipes.

Penstocks of hydroelectric plants are often subject to a rise in water pressure, due to water hammer, which occurs either at the time of filling in, or by suddenly closing the gate of discharge. There being an increase in the water pressure due to its kinetic energy being brought to a sudden stop, the stresses in the shell of the penstock are also increased, but these must not reach the elastic limit of the metal. To remedy this effect, surge pipes are commonly installed along the pipe line, the area of such surge pipes being at least equal to the area of the penstock.

The formula which is ordinarily used to calculate the rise in pressure in a penstock, due to closing of gate is:

$$p = 0.0134\, l \frac{(v_1 - v_2)}{T}$$

in which:

p = rise in pressure, in lbs. per sq. inch,
v_1 = normal velocity of water in penstock, in ft. per sec.,
v_2 = velocity after change in gate opening, in ft. per sec.,
l = length of penstock, in ft.,
T = Time in seconds required to change gate openings.

The coefficient 0.0134 is rather small, and Mr. G. M. Peck[1] proposes that its value be taken at 0.0201, the latter being safer. Mr. Merriman proposes the formula:

$$p = 0.027 \frac{lv}{T}$$

in which: p, l, T are as before and v = velocity of water in feet per second. This formula is materially the same as the one above, the value of v in Merriman's formula corresponding to the value of $(v_1 - v_2)$ in the other formula, and the constant coefficient being taken a little higher to correct for the value of the initial velocity, instead of the mean velocity between the initial and final in a time T.

Sudden variations in the velocity of water in pipes may produce a positive water hammer effect, or a negative one, according to the direction of the variation of the velocity. These phenomena may occur consecutively in a same pipe line, and whereas the positive water hammer may sometimes assume dangerous values, the negative water hammer cannot cause injury, because, if vent pipes are provided, the pressure inside the pipe cannot become less than the atmospheric pressure. Mr. L Allievi[2] gives the following formula for negative water hammer, in order to determine the time T of opening so that the maximum decrease in pressure shall not exceed a given value $(1 - s)$.

The formula is:

$$T = 2 \frac{l v' \sqrt{s}}{g H_0 (1 - s)}$$

[1] Engineering News. Aug. 11, 1910.
[2] Revue de Mécanique, Paris. January-March, 1904.

in which

T and l = are as before, l being in meters,
g = constant of gravity, = 9.81,
H_0 = head of water, in meters,
v' = velocity of water in pipe, in meters per second,
$s = \frac{H}{H_0}$ where H is the resulting pressure in meters.

61. Wooden Stave Pipe.

This sort of pipe has been extensively used of late. It presents serious advantages, and replaces economically open wooden flumes. In the continuous stave type, staves are laid side by side, and being of different lengths, all joints are staggered. Each stave is butted against the one immediately preceding it, while slits at the end receive a metallic tongue. The staves are held firmly in place by steel rods solidly clinched until every stave is well seated. See fig. 76.

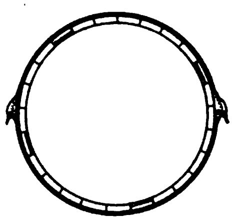


Fig. 76.
Wooden Stave Pipe.

The essential condition to insure long life of the wooden staves is that they must be kept constantly saturated. To accomplish this, it is best to bury the pipe under ground, and make the wood of such thickness as to be always water soaked.

The steel rods or bands are placed upon the pipe for the purpose of resisting the water pressure, water hammer, and the swelling power of the wood, once it is saturated. The bands are preferably of round steel, because of their adaptability of embedding themselves in the wood as it swells and at the same time, they offer less surface to corrosive influences. Their spacing, diameter, and bearing surface upon the pipe will be proportioned with due regard to specific requirements. The ends of the bands are secured together by means of a malleable iron shoe, so shaped that a close fit upon the exterior of the pipe is obtained, and composed of two shoulders, the interior one for the head of the bolt, the exterior one for the nut. In this way, the rod lies in a plane perpendicular to the axis of the pipe, and the strain of the rod produces a straight pull.

With perfect material and superior workmanship, a wooden pipe may be manufactured to withstand a pressure of 300 ft. head.

It is claimed that the price of wooden stave pipe is from 20% to 60% lower than that of metal pipe, and other essential advantages are that it is light, strong and easy to handle. Its inner surface remains clean from tuberculations which diminish the carrying capacity of metal pipes. It has been observed that the carrying capacity of wooden pipe increases with age as the surface gets smoother, while the friction of the metal pipe increases.

62. Reinforced Concrete Pressure Pipes.

These are chiefly used where it is desired to convey large quantities of water at low pressure. As $\frac{3}{16}$ in. plate is the smallest gage used for steel pipes, it is clear then, if $\frac{3}{16}$ is too strong a plate to perform the work eco-

nomically, that is to say, if this will resist a pressure that will never occur, there is evidently a waste of metal. In some instances, reinforced concrete pipe is subjected to a pressure of 75 ft. of water. Reinforced concrete pipe consists in the incorporation of a steel structure composed of circular or longitudinal bars into the concrete. These reinforcing steel bands are calculated to carry all the stresses due to water pressure, fill or span, and the concrete is simply considered as a fill-in material. It may be remarked however, that in the case of a concrete pipe being supported by piers, the upper part of the pipe, that above the neutral axis, may be allowed to carry compressive stresses. An illustration of this case is submitted hereafter. The area of necessary steel may be calculated by comparing the circular reinforcement to an equivalent full shell, then numbering and sizing the circular bands in such a way that the equivalent area of metal is obtained.

The longitudinal reinforcement, the area of which amounts generally to $\frac{1}{4}$% of the concrete area, is designed to take care of temperature and shrinkage stresses. The circular reinforcement may be spirally wound bars or individual hoops. In the latter case, the bars are overlapping at least 30 bar diameters and staggered around the circumference of the pipe.

As has been said before, the concrete is simply a fill-in material. If its thickness varies according to the class of mixture of cement and the pressure of water, it is not necessarily a function of the diameter. It is nothing but a question of spacing the steel bars, and leaving enough border margin to secure a good protection of the metallic parts. In order to have an absolute water tightness, it is best to give a finish coat of mortar or asphalt.

A head of 75 to 100 ft. may safely be supported by a pipe of the sort described, and if silt bearing water is conveyed it is best to place the pipe at a slope of at least 0.01 and to locate a sump hole and blow-off at the point of origin of the following steel pipe.

Reinforced concrete pressure pipes are either built in place or constructed at the manufacturers' plant, the latter system being employed and usually gives better results for pipes of small diameter.

63. Spacing of Piers for Reinforced Concrete Pipes.

Mr. P. Caufourier, in the *Génie Civil,* expounds the following method for calculating the proper relations in the problem of pier spacing.

The steel reinforcement is composed of circular and longitudinal bars. As the circular bars are calculated to resist the water pressure and such pressure as may cause deformation (fill, etc.), they are subject to the analysis which has already been given in paragraph 47.

However the longitudinal reinforcement must take care of the bending effort of the pipe between supports. These longitudinal bars, uniformly spaced, will be replaced by another fictitious pipe, the steel shell of which shall have an equivalent area.

Let

r = mean radius of pipe, see fig. 77,
e = thickness of concrete shell,
ε = thickness of shell of equivalent fictitious pipe,

NN' = neutral axis defined by angle α,
α = angle limited by radius ON and the plane of bending,
m = ratio of coefficients of elasticity of steel and concrete.

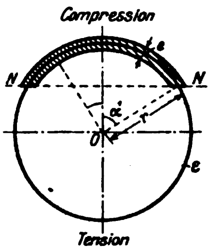


Fig. 77.

Tension shall not exist in concrete, so that e does not have to be considered below the neutral axis.

The position of the neutral axis is determined by the expression:

$$\tan \alpha - \alpha = m \pi \frac{\varepsilon}{e} \quad \ldots \ldots \quad (1)$$

which may easily be resolved by means of tables of arc lengths and natural functions of tangents, finding such value for α as will make $(\tan \alpha - \alpha)$ equal to $m \pi \frac{\varepsilon}{e}$.

The moment of inertia with respect to the neutral axis just determined is

$$I = 2 r^3 m \varepsilon \left[\frac{\pi}{2} + \pi \cos^2 \alpha \right] + 2 r^3 e \left(\frac{\alpha}{2} - \frac{3}{4} \sin 2\alpha + \alpha \cos^2 \alpha \right) . \quad (2)$$

The maximum elastic stresses are then found by the formulas: (in which M_x = maximum bending moment):

$$R_{tens.} = \frac{m M_x r (1 + \cos \alpha)}{I} \quad \ldots \ldots \ldots \quad (3)$$

$$R_{comp.} = \frac{m M_x r (1 - \cos \alpha)}{I} \quad \ldots \ldots \ldots \quad (4)$$

$$R_{concrete} = \frac{M_x r (1 - \cos \alpha)}{I} .$$

From these formulas:

$$\frac{R_{concrete}}{R_{tens.}} = \frac{1 - \cos \alpha}{m (1 + \cos \alpha)} .$$

This gives α, knowing $R_{tens.}$, $R_{concr.}$.

This value of α substituted in equation (1) gives the thickness ε or, which is the same thing, the percentage of reinforcement.

64. Numerical Examples for Reinforced Concrete Pipes.[1]

To design a reinforced concrete pipe of 5 ft. inside diameter, maximum hydrostatic pressure (water hammer effect included) is 50 ft. Pipe will be laid in wet sandy soil, and covered with a 4 ft. fill over the top. It will be supported on piers across a declivity, at intervals to be determined.

There will be two layers of reinforcement, one near the outer and one near the inner faces of the pipe.

It will be assumed that the thickness of the concrete shell is 6 inches. If the steel alone must resist the pressure of the water, considering that the

[1] All calculations have been made by means of the slide rule.

limiting stress be 12 500 lbs. per sq. in., the thickness of shell of an equivalent full steel pipe would be accorting to formula (1) on pag. 96

$$\varepsilon = \frac{50 \times 5}{4800 \times 0.75} = 0.052''$$

which corresponds to 0.63 sq. in. of steel per lineal foot of pipe.

Stresses due to fill. The pressure due to fill is considered as being uniform, and average depth of fill 6 feet.

$$w_v = 100 \times 6 = 600 \text{ lbs. per sq. ft.}$$

The intensity of the horizontal pressure being taken as one third of the vertical, and the approximate depth to the center line being $6\frac{2}{3}$ feet, the average horizontal pressure will be:

$$w_h = \frac{100 \times 6.66}{3} = 222 \text{ lbs. per sq. ft.}$$

By substitution

$$M_b = -M_a = \left(1 - \frac{w_h}{w_v}\right) w \frac{r^2}{4} = \left(1 - \frac{222}{600}\right) 600 \times \frac{6.25}{4} = 590.63 \text{ ft. lbs.}$$
$$= 7087.5 \text{ in. lbs.}$$

The area of reinforcement will be determined by the formula

$$M = sq \times 0.86\,d$$

where s is the unit stress of steel.

$$q = \frac{M}{s \times 0.86\,d}.$$

Assuming $d = 5''$

$$q = \frac{7087.5}{12{,}500 \times 0.86 \times 5} = 0.132 \text{ sq. in.}$$

This amount of steel is required along both faces. Having determined the amount of steel that will resist the water pressure, and the amount that will resist the stress due to fill, the total amount required will be:

$$0.63 + (2 \times 0.132) = 0.894 \text{ sq. in.}$$

It will be necessary then to use $\frac{5}{8}''$ round bars, spaced 4 inches center to center.

Longitudinal reinforcement between supporting piers. Calculation of span.

As this same pipe is supposed to cross a declivity of the ground, and be supported by piers, it is necessary to apply the formulæ developed by Caufourier for determining the proper spacing. The thickness, ε, of an equivalent fictitious shell is 0.052 in. (It is assumed that the longitudinal reinforcement is of the same area as the transverse reinforcement.) The thickness, e, of the concrete shell is 6 in. The position of the neutral axis will be determined by proper substitution in formula (1) page 96. It is found that the value of α that satisfies the equation is approximately 50 deg. 20 min., for which the arc value is 0.878 and tan α is 1.206.

In the formula

$$\frac{R_{concrete}}{R_{tension}} = \frac{1 - \cos\alpha}{m(1 + \cos\alpha)}$$

let $R_{tension} = 12{,}500$ lb. per square inch,

$$m = \frac{29{,}000{,}000}{2{,}400{,}000} = 12 \text{ (approximately)}$$

$$\cos \alpha = 0.638.$$

Solving for $R_{concrete}$, a value is found of approximately 230 lbs. per square inch.

Remembering that

$$\varepsilon = 0.052''; \qquad e = 6'';$$
$$\alpha = 0.878; \qquad \cos^2 \alpha = 0.407;$$
$$\sin 2\alpha = 0.983; \qquad r^3 = (2.75 \times 12)^3$$

the moment of inertia will be obtained by formula (2), on page 96:

$$I = 2(2.75 \times 12)^3 \cdot 12 \cdot 0.052 \left[\frac{\pi}{2} + \pi \cdot 0.407\right]$$
$$+ 2(2.75 \times 12)^3 \cdot 6\,[0.439 - 0.737 + 0.358] = 153{,}562 \text{ in}^4.$$

It is assumed that the pipe is acted upon as a uniformly loaded beam with fixed ends. The maximum bending moment is given by

$$M_x = \frac{W l}{12}$$

where W is taken as 2500 lbs. per linear foot (weight of pipe shell and water within).

Then

$$M_x = 2500\, l^2 \quad \text{inch-lbs. } (l \text{ being in feet}).$$

Substituting this value of M_x in formula (3) and solving for l gives $l = 97$ feet.

Knowing the safe span, the compressive stress for steel will be found by proper substitution in formula (4). In the present case compression is very small, taking into account the considerable thickness of concrete. $R_{comp.}$ is found to be 2040 lb. per square inch.

65. Self-supporting Penstock Bridge.

This bridge constructed in connection with the water power plant of La Praz, near Modane, carries the water across a span of fifty meters, and is interesting for the fact that the diameter of the pipe which forms the bridge is of the same diameter as that of the rest of the pipe line, and the penstock bridge entirely supports itself, there being no intermediary pier.

The design was made[1] by Messrs. Seyrig and Rateau, and the following data is taken from a paper by Mr. Laponche.

The conditions were as follows:

Span, 50 meters
Radius of arc, 100 meters;
Inside diameter of pipe: 2.40 m
Thickness of plate 15 mm; the extreme plates at the piers having a thickness of 16 mm.

[1] Note sur une conduite d'eau. M. A. Laponche in "Bulletin et Comptes Rendus de la Société de l'Industrie Minérale." Vol. 13; p. 793.

The arch was considered as having fixed ends.

The elastic forces which are developed in the metal result from four causes:

a) hydrostatic pressure due to head of water;
b) the weight of conduit and aqueous contents;
c) expansion and contraction due to change of temperature;
d) wind stresses.

Wind stresses were neglected in this particular case, the bridge being situated in a narrow valley, and its direction parallel to that of the wind. The influence of temperature was also neglected, the temperature of the water varying very little and because the field work was done in autumn, when the temperature of the atmosphere was nearly equal to that of the water.

The elastic forces resulting from hydrostatic pressure were found to be 5.45 kg per sq. cm.

The load per lineal meter is as follows:

Weight of metal	= 900 kg
Weight of water	= 4500 kg
Total	= 5400 kg.

The load, referred to a lineal foot of the arc, being 5400 kg, if referred to a lineal foot of the chord of the arc, varies from 5400 kg at the crown to

$$5400 \frac{1}{\cos 14^0\,30'} \text{ (at the springing lines)}$$

that is to say the load varies from 5400 kg to 5575 kg.

In order to simplify the calculations, a uniform load of 5700 kg per lineal foot was considered (weight of rivets etc. included).

The elastic forces which result from the load may be determined from the components along the normal and along the tangent of the mean fiber; these components are deducted from the knowledge of:

1) tension, or compression, of the mean fiber,
2) shearing stress,
3) bending moment,
4) moment of torsion.

The complete analysis is given here, as it may prove of interest, to designers of self supporting penstock bridges.

Let:

M = bending moment of the center of gravity of the section,
N = compression of mean fiber,
I = moment of inertia of section,
Ω = a section perpendicular to mean fiber,
v = distance of fiber of section Ω to mean fiber.

The component of elastic forces, normal to the section, has for value

$$n = \frac{M v}{I} + \frac{N}{\Omega}.$$

For any given section, the bending is a maximum, either for the intrado or extrado fibers; it is therefore only necessary to study these two

fibers. The expression of N can be simplified by the introduction of the notion of reciprocal points.

Being given (fig. 78) one section $A' A''$, a reciprocal point of the intrado fiber of such a section is a point a' at a distance u' from the mean fiber in such a way that

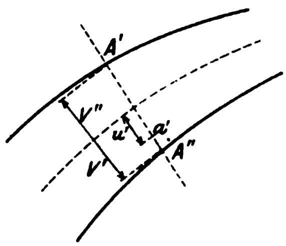


Fig. 78.

$$u' v' = r^2$$

r^2 being the square of the radius of gyration of the section.

In the same way the reciprocal point of the extrado fiber is given by the relation:

$$u'' v'' = r^2$$

If we call M' and M'', the moments of the forces referred to the reciprocal points acting on the arcs, we obtain the elastic forces developed in the intrado and extrado fibers by means of the formula:

$$n' = \frac{M' v'}{I}$$

$$n'' = \frac{M'' v''}{I}.$$

The tangential components of the elastic forces are composed of two parts:

1 st: The one produced by the shearing force T expressed by

$$t'_1 = \frac{T}{\Omega}.$$

2 nd: That one produced by the moment of torsion M_t which has for value:

$$t'_2 = \frac{M_t v}{I_p}$$

I_p being the polar moment of inertia of the section.

The moment of torsion is zero in the present case because all the acting forces are situated in the plane of the mean fiber of the arc; therefore:

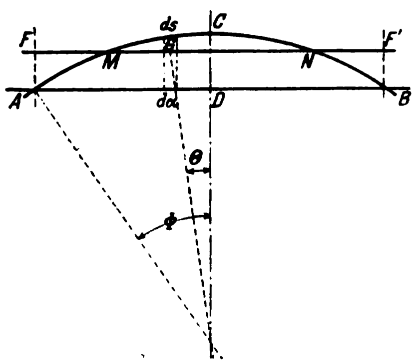


Fig. 79.

$$t'_2 = 0$$

Calculations of the thrust. For an arch with fixed ends, the thrust is given by the formula

$$Q = \frac{\int_A^B (My + r^2 S_x) \frac{ds}{I}}{\int_A^B (y^2 + r^2) \frac{ds}{I}}$$

in which:

M = bending moment produced at any point G of the mean fiber of the arc by the given forces (fig. 79);

S_x = sum of the horizontal projections of the forces located to the left of each point G;

y = ordinate of the point G;

ds = indefinitely small element of the closing line of the arc which contains the point G.

The closing line of the arc is determined by the condition that the fictitious forces, equal to $y\frac{d_s}{I}$, be in equilibrium. If I is constant and if we call dx the horizontal projection of ds we have

$$dx = ds \cos \Theta.$$

Θ varies from 0° to 14° 30′; therefore $\cos \Theta$ varies from 1 to 0.97.

In making a maximum error of 3% it can be admitted that

$$dx = ds$$

and we are brought back to the construction of a line, so that the forces ydx are in equilibrium. The closing line of the arc is then a line such that

$$\text{area } AFM + \text{area } BF'N = \text{area } MCN$$

or

$$\text{area } AFF'B = \text{area } AMCNB$$

that is to say

$$AF \cdot AB = \left[\frac{\pi \Phi}{180} - \sin \Phi\right] \frac{R^2}{2} = 95 \text{ sq. meters}$$

from which

$$AF = \frac{95}{50} = 1.90\,\text{m}.$$

As the arc is submitted only to the action of vertical forces we have

$$S_x = 0$$

Suppose that the forces applied to the arc are reduced to a weight P equal to unity, dividing the arc in a definite number of parts equal to Δs; as I is constant, we have approximately:

$$Q = \frac{\sum_A^B M y}{\sum_A^B (y^2 + r^2)}.$$

The numerator of this expression represents the sum of the moments produced at the point of application of the weight $P = 1$ multiplied by the vertical forces y applied at the bases of the ordinates y of the different division points. This sum of moments has been determined graphically (fig. 83) by the construction of the funicular polygon AM of the forces y with a polar distance $\Omega_1 A$ equal to the rise CD of the arc.

Calling β the ordinate of this funicular polygon which corresponds to the point of application of the weight $P = 1$, we have:

$$\sum_A^B My = \beta \cdot CD.$$

The denominator of the expression of the thrust may be written:

$$\sum_A^B (y^2 + r^2) = \sum_A^B Y\,Y$$

in which $Y\,Y$ are the ordinates of a curve which can be called *transformed curve of the mean fiber*, and which may be deduced by the relation:

$$Y^2 = y^2 + r^2.$$

The denominator of the fraction which expresses the thrust is therefore a constant, which represents the sum of the moments, (with respect to the x axis) of the horizontal forces equal to the ordinates Y of the transformed curve, applied at the corresponding points of the latter.

This sum of moments has been determined (fig. 83) by the construction of the funicular polygon $\Omega_2 R_1 R_2 R_3$ of one half of the forces of Y with a polar distance $\Omega_2 P$ equal to the rise CD of the arc. Calling λ the portion mn of the x axis contained between the last side of this funicular polygon and the vertical at the crown of the arc, we have:

$$\sum_A^B (y^2 + r^2) = \sum_A^B Y Y = 2\lambda \cdot CD.$$

The thrust can therefore finally be written:

$$Q = \frac{\beta \cdot CD}{2\lambda \cdot CD} = \beta \cdot \frac{1}{2\lambda}.$$

If instead of only one force $P = 1$ we have a series of forces $P_1 P_2 .. P_n$ applied to the arc, we will have:

$$Q = \frac{1}{2\lambda} \sum P_n \beta_n.$$

The curve, the ordinates β of which permit the determination of the thrust, is called the line of thrust of the arc.

In this particular case, the loads P have a constant value and we can write

$$Q = \frac{P}{2\lambda} \sum \beta.$$

The chord of the arc has been divided in twenty equal parts to which correspond equal loads of 14,250 kilogr. The sum of the ordinate β measured on the drawing to the scale of length has for value 25.8 m. The quantity λ measured in the same way gives 0.61 m. (to facilitate the graphical constructions, the curve, the ordinates of which are equal to 3 Y, has been drawn; it results therefrom that the quantity $\lambda' = m\,n$ (183 mm) measured on the drawing is three times larger than the quantity λ to be substituted in the formula). We have therefore:

$$Q = \frac{14250}{2 \times 0.61} \times 25.8 = 301,000 \text{ kg}.$$

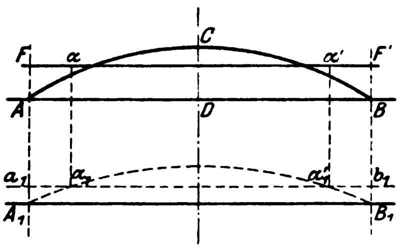


Fig. 80.

Construction of the polygon of pressures. The polygon of pressures is a funicular polygon of the forces applied to the arc drawn with a polar distance equal to the magnitude of the thrust. In order to determine it, it is necessary to know one point thereof. Construct a funicular polygon of the forces applied to the arc, the polar distance being Q (fig. 80); let $A_1 B_1$ be this polygon, and $a_1 b_1$ the closing line of this polygon; project the points $\alpha_1\ \alpha_1'$ over the closing line of the arc at $\alpha\,\alpha'$; the two points α and α' are points of the pressure polygon.

This construction results from the fact that the bending moment of a point of the mean fiber has for value:

$$M = Q\ (z'' - y) = Qn$$

in which:

z'' = ordinate of the polygon, $A_1\ B_1$, measured from $a_1\ b_1$;
y = ordinate of the mean fiber measured from the closing line;
n = ordinate of the pressure polygon, measured from the mean fiber of the arc.

This polygon is obtained immediately in measuring the ordinates of the polygon $A_1\ B_1$ starting at $F\ F'$ and measured to $a_1\ b_1$.

Construction of reciprocal points. According to the definition, the locii of the reciprocal points for an arc of constant section, are curves parallel to the mean fibers; in the present case they are circles.

The square of the radius of gyration is:

$$r^2 = \frac{I}{\Omega} = \frac{\frac{\pi}{64}(D^4 - d^4)}{\frac{\pi}{4}(D^2 - d^2)} = \frac{1}{16}(D^2 + d^2) = 0.725\,.$$

Therefore the radii of the circles, locii of the reciprocal points, are:

$$R' = R - \frac{r^2}{V'} = 100 - \frac{0.725}{1.20} = 100 - 0.605 \text{ (intrado fibers)},$$

$$R'' = R + \frac{r^2}{V''} = 100 + \frac{0.725}{1.20} = 100 + 0.605 \text{ (extrado fibers)}.$$

The drawing of these circles shows that they cut the pressure polygon at the points 1′, 2′; 1″, 2″.

Determination of the bending moments: The bending moment for the different sections is obtained by multiplying the thrust by the vertical distance contained between the polygon of pressures and the reciprocal point of the intrado or extrado fiber.

The sign of the bending moment is determined by the position of the reciprocal point considered with respect to the polygon of pressures, the moment being positive, if the reciprocal point is above the polygon of pressures.

Table XXXII is deducted from the graphical construction, and gives bending moments for sections 5 meters in length. The drawing shows that the bending moment is zero at the points 1′ and 2′ for the intrados, at the points 1″ and 2″ for the extrados. It is seen, besides, that there are two maximum values, one at the crown, the other at the springing lines.

Table XXXII.

Distance of considered section to abutment *A*		0	5 m	10 m	15 m	20 m	25 m
Distance of reciprocal point to polygon of pressures	Intrados	+ 0.98	+ 0.1	— 0.60	— 1.13	— 1.45	— 1.64
	Extrados	+ 2.30	+ 1.40	+ 0.62	+ 0.10	— 0.20	— 0.40
Bending Moment $M = Qn$	Intrados	+ 295 *MT*	+ 30	— 180	— 340	— 435	— 492
	Extrados	+ 690 *MT*	+ 420	+ 186	+ 30	— 60	— 120

Calculations of the elastic forces normal to the sections. These are deduced from the bending moments by the formula:

$$n = \frac{Mv}{I}$$

in which I has for value

$$r^2 \Omega = 0.0825$$

if the gross section of the plate is considered; taking care of rivet holes, the net section is equal to $^3/_4$ of the actual or gross section, so that:

$$I' = 0.75 \times 0.0825 = 0.061875 .$$

Table XXXIII gives the elastic forces per square millimeter for the gross and net sections, the lower figures corresponding to the last hypothesis.

Table XXXIII.

Distance of the section to abutment A		0	5 m	10 m	15 m	20 m	25 m
Normal elastic forces	Intrados	+ 4.300 5.750	+ 4.375 5.820	— 2.625 3.500	— 4.950 6.600	— 6.330 8.400	— 7.180 9.600
	Extrados	+ 10.000 13.400	+ 6.120 8.200	+ 2.710 3.600	+ 4.375 5.850	— 0.875 1.170	— 1.750 2.330

Tangential elastic forces. These are reduced to the forces produced by the shear. For a given section, the shearing stress is the component along the normal to the mean fiber, of the resultant transfer of all the forces situated to the left of the considered section:

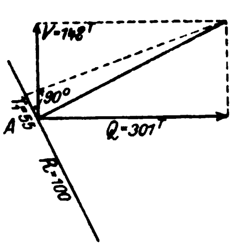


Fig. 81.

1st: At the springing lines. The resultant of transfer is the resultant of the thrust $Q = 301^T$ and of the vertical reaction $V = 142^T$ (fig. 81).

The shearing stress is found to be:

$$T_1 = 55^T$$

2nd: At the crown. At this point, the tangent to the polygon of pressures is horizontal; therefore the resultant of transfer is also horizontal; consequently, the shearing stress is zero at the crown.

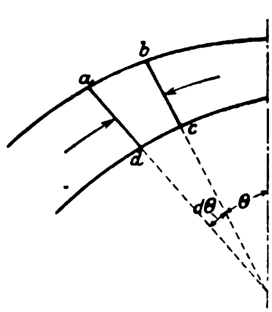


Fig. 82.

It results from the above that the tangential elastic forces vary from $\frac{0}{\Omega}$ at the crown to $\frac{55^T}{\Omega}$ at the springing lines, in other words, from 0 to

$$\frac{55{,}000}{114{,}000} = 0.48 \text{ kg},$$

these elastic forces can therefore be neglected.

The elastic forces produced by hydrostatic pressures take place only when the pipe is under pressure.

Suppose an element $abcd$ corresponding to an indefinitely small length ds of the mean fiber (fig. 82), the volume $abcd$ is submitted to the action of the following forces:

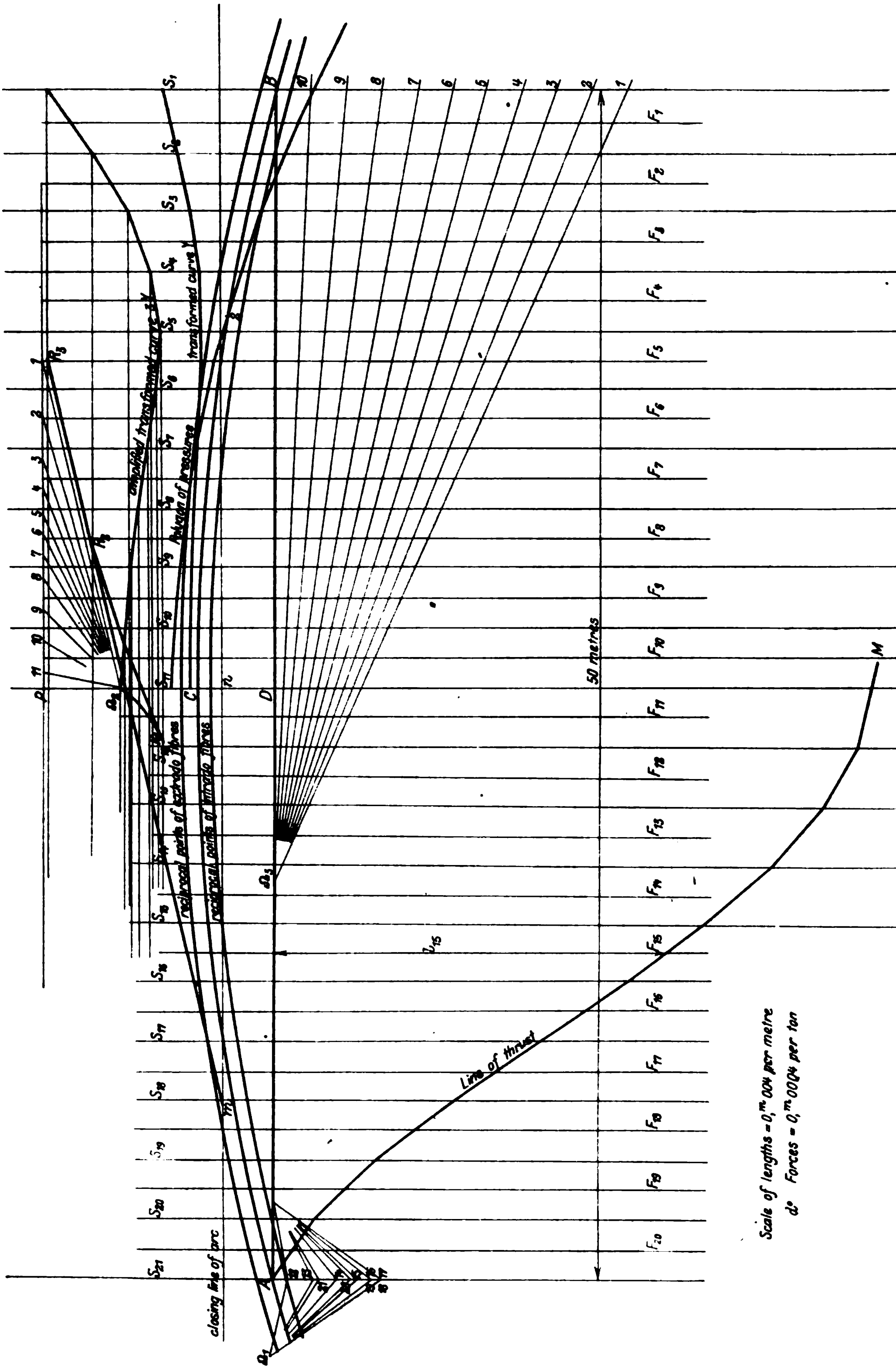

Line of thrust
50 metres
amplified transformed curve
transformed curve
Polygon of pressures
reciprocal points of extrados fibres
reciprocal points of intrados fibres
closing line of arc
Scale of lengths = 0,m 004 per metre
do Forces = 0,m 0004 per ton

Fig. 83.

1st: Gravity;

2nd: The reactions of the pipe; equal and opposed to the hydrostatic pressure;

3rd: The pressures on the faces ad and bc which can be substituted by their resultant directed along the bisectrix of the angle $d\Theta$. The resultant of the pressure on the faces $a\,d$ and $b\,c$ has for value:

$$2\,\omega\,\Omega' \sin\frac{d\Theta}{2};$$

in which

ω = interior pressure per unit of area; and

Ω' = interior section of the pipe; the vertical component of this resultant is:

$$2\,\omega\,\Omega' \sin\frac{d\Theta}{2}\cos\Theta; \quad \text{or} \quad \omega\,\Omega' d\Theta \cos\Theta.$$

Table XXXIV.

Distance of the considered section to abutment A			0	5 m	10 m	15 m	20 m	25 m
Elastic forces produced by the hydrostatic pressures			5.450	5.450	5.450	5.450	5.450	5.450
			7.300	7.300	7.300	7.300	7.300	7.300
Elastic forces produced by gravity,	Pipe filled with water but not under pressure.	Intrados	+ 4.300	+ 4.375	— 2.625	— 4.950	— 6.330	— 7.150
			+ 5.750	+ 5.820	— 3.500	— 6.600	— 8.400	— 9.600
		Extrados	+ 10.000	+ 6.120	+ 2.710	+ 4.375	— 0.875	— 1.750
			+ 13.400	+ 8.200	+ 3.600	+ 5.820	— 1.170	— 2.330
	Id. under pressure.	Intrados	+ 2.040	+ 2.070	— 1.190	— 2.350	— 3	— 3.400
			+ 2.720	+ 2.750	— 1.580	— 3.140	— 4	— 4.520
		Extrados	+ 4.750	+ 2.900	+ 1.280	+ 2.070	— 0.415	— 0.830
			+ 6.350	+ 3.870	+ 1.710	+ 2.750	— 0.552	— 1.110

The horizontal component is

$$2\,\omega\,\Omega' \sin\frac{d\Theta}{2}\sin\Theta; \quad \text{or} \quad \omega\,\Omega' d\Theta \sin\Theta.$$

As Θ varies from 0^0 to $14^0\,30'$, $\cos\Theta$ varies from 1 to 0.97.

It can therefore practically be admitted that the vertical component is always equal to $\omega\,\Omega' d\Theta$ and the error which is introduced is 3% at the maximum; this component however is directed to the opposite side of the center of curvature of the arc. It results therefrom that when the arc is loaded, it can be considered as being submitted to the action of ascending vertical forces having for value $\omega\,\Omega' \mathrm{d}\Theta$ for the length ds.

If instead of an indefinitely small length ds, definite lengths Δs equal to $1^{m}.000$ are taken, we will have:

$$\Delta\Theta = 34.$$

In this case the value of the ascending vertical forces will be

$$\Phi = 2 \times 0.068 \times 4{,}500{,}000 \times \sin\frac{34}{2} = 3000\,\text{kg}.$$

It results therefrom that if the water in the pipe is under pressure, the effect of gravity is diminished and the vertical forces acting on the arc are reduced to

$$5700 - 3000 = 2700 \text{ kg}.$$

As the elastic forces are proportional to the acting forces, it results from the above, that the elastic forces as determined before be multiplied by the ratio $\frac{2700}{5700}$.

Table XXXIV resumes the different hypotheses that can be considered, and the calculations have been made for the gross and net sections.

Remark. It has been shown that water under pressure in the pipe diminishes the effect of gravity. Let us determine the radius of curvature R of the arc, so that the total weight, or the weight contained within the pipe be equilibrated by the hydrostatic pressure.

Consider an indefinitely small element *abcd* the weight of which is $1000\,\Omega' ds$; the resultant of the action of the water against the two plane faces of this element is

$$2 \times 0.068 \times \Omega' \sin \frac{d\Theta}{2}$$

and, replacing the sine by the arc, $0.068\,\Omega' d\,\Theta$.

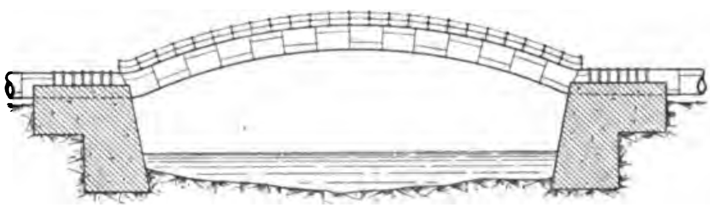


Fig. 84. La Praz Penstock Bridge.

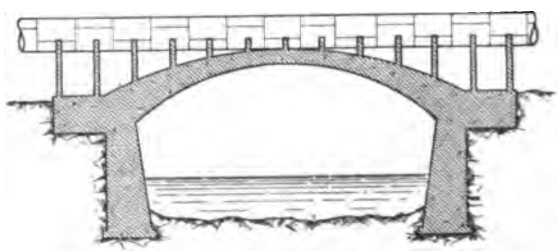


Fig. 85. Concrete Penstock Bridge.

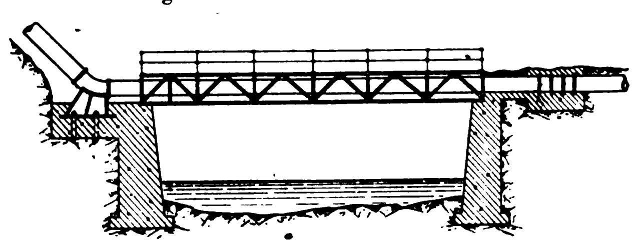


Fig. 86. Typical Steel Penstock Bridge.

Admitting that this force be always vertical the condition of equilibrium will be

$$1{,}000\,\Omega' ds = 68{,}000\,\Omega' d\,\Theta$$

or

$$\frac{ds}{d\,\Theta} = R = 68$$

in other words, the *weight of the water contained within the arc, will be equilibrated by the hydrostatic pressures, if the radius of curvature of the arc is equal to the head of water in lineal measure.*

The general layout of the La Praz penstock bridge is shown in fig. 84. Penstock bridges of other designs are illustrated in fig. 85 and 86.

Chapter V. Dams.

66. Earthen Dams.

These dams are generally constructed where the soil is little resistant, or of a low safe bearing value. They are made of specially selected natural materials found on the site, or of a mixture of loam, clay and rock. The safety of an earthen dam depends on its water-tightness. With this object in view, it is best to construct dams having a masonry or concrete core wall in the center, with puddle or selected material placed upon its water face. Percolation through the dam may also be eliminated by covering the upstream face with plank, made water tight by means of asphaltum or tar.

English dams are generally provided with a puddle core, the water face being of any material found on the site, and between the face and the puddle core, especially selected material is placed. In many instances, other material has been used, such as timber, steel and reinforced concrete, which are as efficient as the ordinary earthen dam. In case of timber being used, the timbers are placed alternately parallel and perpendicular to the direction of the stream, the spaces being filled with earth and stone.

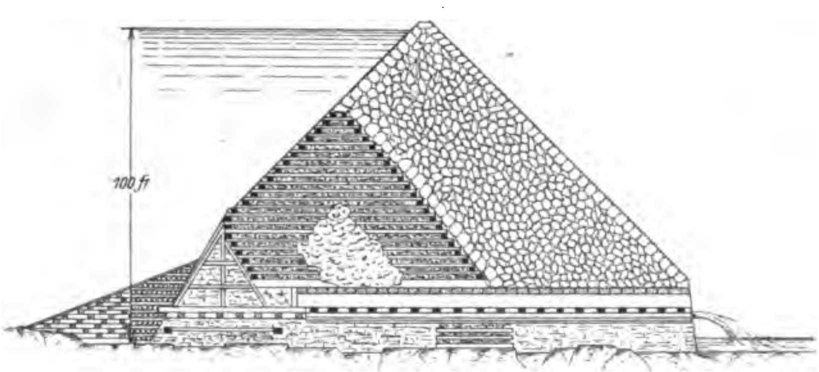


Fig. 87. Cross Section of the Bowman Rock-Fill Dam.

An example of a timbercrib rock filled dam is given in fig. 87 illustrating the Bowman type. This particular dam is 100 ft. high, and consists of a timber crib of unhewn cedar and tamarack logs, notched and bolted together, then filled with small stone. The slopes on each side are 1 to 1 and the face was made with pine planking, laid horizontally. The slope adopted in the construction of earthen dams is generally 2 to 1 on the exterior and 3 to 1 on the interior, and when the earth is laid it should be moist so as to pack solidly, however this moisture should not be excessive.

Fig. 88 illustrates a rock fill dam with steel core. This consists of a steel web-plate, varying in thickness from 0,33 in. at the bottom to 0.25 inch at the top. The plate is covered with a coat of Alcatraz asphalt, over which a layer of burlap is placed. A second coating of asphalt has been applied over the burlap, and the whole imbedded in a masonry wall 2 ft. thick, the masonry consisting of Portland cement laid with rubble.

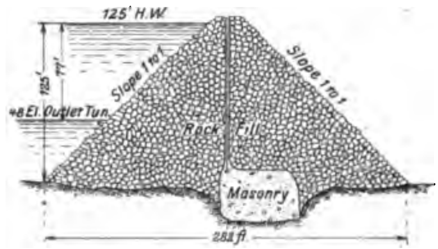


Fig. 88. Rock Fill Dam with Steel Core.

Another type of dam usually known as the hydraulic fill dam, consists in an embankment of earth, sand and gravel sluiced into position by flowing water. A dam built by this method is the Necaxa dam in Mexico, fig. 89, where construction was done as follows: The ground was first cleared and stripped, a trestle to support the flume was erected, and low earth dikes were made at the upstream and downstream limits of the fill, to hold the mud and water. The material was then sluiced in, the pipes discharging near the embankments, so that the boulders and gravel were deposited on the faces and in the fine mud in the center of the dam. The dikes were raised as the dam was filled, and the water spilled over the upstream face into the pond. During the construction stage, the water of the river passed through the discharge gates, which were made large enough for the purpose. A spillway was provided over a neck of rock to the north of the dam.

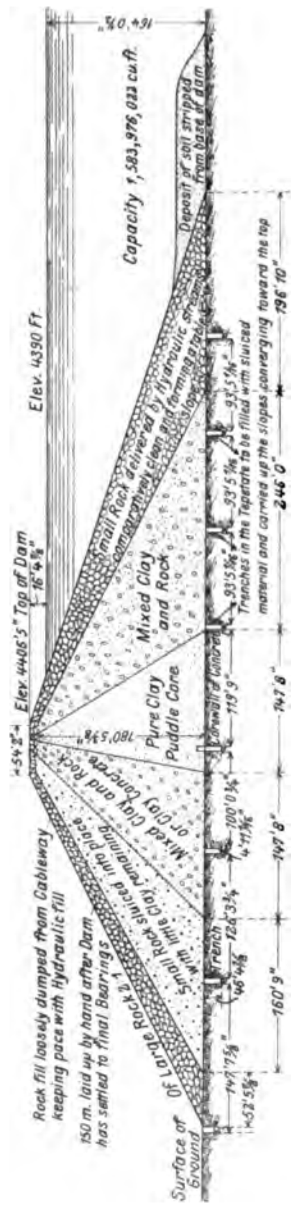


Fig. 89. Typical Section of Necaxa Reservoir Dam.

The most important factor influencing the safety of a dam of any of the types here described, consists as was said before, in its water tightness, which may be brought to its maximum by the proper arrangement of outlet pipes to regulate and control the stored water. These pipes should not be built through the body of the dam, as they cannot always remain as originally laid, but sooner or later the pipes become fractured, or the joints are drawn, with the result that the water will have full play and the destruction of the dam will inevitably result.

In several cases, outlet arrangements consist in a tunnel driven through solid rock, passing around one end, and entirely disconnected from the embankment. The tunnel is made to discharge flood water, in which case there need not be a spillway.

It will be of interest to note fig. 90, which gives a section of a dam built for rivers exposed to sudden floods, and the bed of which consists of sand. This particular dam was built on the Arkansas river and has successfully resisted the effects of several severe floods.

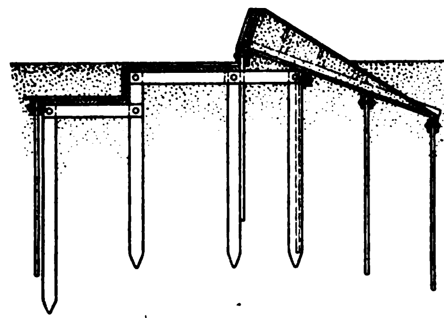
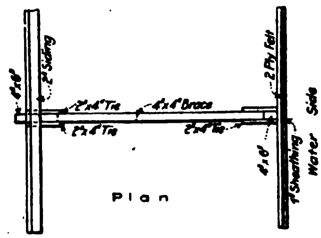


Fig. 90. Dam built on Sand Foundation.



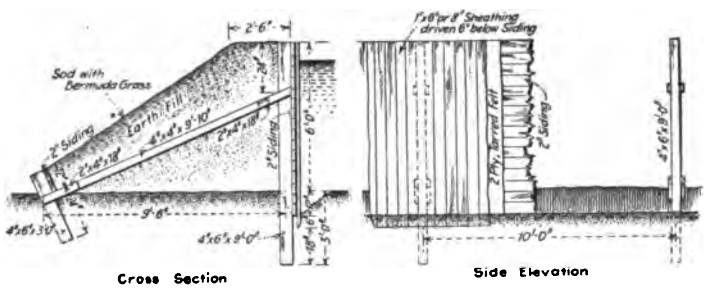


Fig. 91. Type of inexpensive Earth Dam.

Another efficient cheap dam, designed by H. D. Mendenhall is illustrated in fig. 91.

The dam as designed and erected is simply an impermeable facing built on a timber frame work and filled with earth. The frame work for the typical 6 ft. dam consists of 4 × 6 in. pine posts 9 ft. long, sunk 3 ft. into the ground and spaced 10 ft. apart. At a point 9 ft. 6 in. horizontally back from these posts are other posts 4" × 6" × 3 ft. long, sunk 2 ft. into the ground and inclined away from the front posts about 30° from the vertical.

67. Timber Dams.

These structures are very often built as measuring weirs; and are also recommended when the work is to be of a temporary nature. They are used generally for the purpose of diverting the water.

The foundation of a timber dam may be of almost any material, whether rock, gravel, or sand; in the latter case stones may be dumped over the area upon which the dam is to be built, and when the river bottom is too soft, piling is resorted to, and underflow is prevented by sheet piling.

Timber dams may be divided in three classes:

1st: the crib dam;
2nd: the rafter and strut framed dam; and
3rd: the Beaver type.

The crib dam consists of cribs built of logs or squared timber, and is in most cases filled with stone, the timber parts acting rather as a binder, it is also designed with a view to water tightness. If a crib dam is not filled with stone, the up-stream slope should be long, so that the stability will be unquestionable, and failure due to lack of resistance to shearing forces will be avoided.

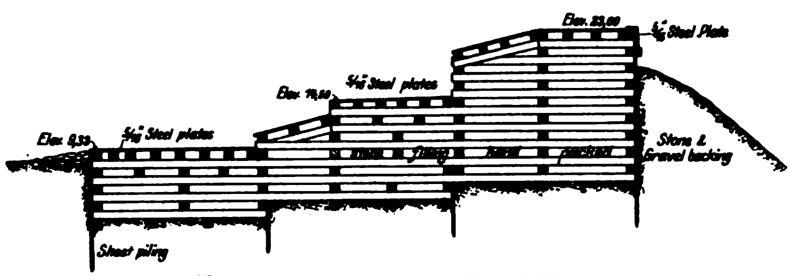


Fig. 92. Timber Dam at Sewall Falls.

Fig. 92 is a section of the Sewal Falls crib dam, built across the Merrimac River. The crib work is formed of 10 and 12 inch square timbers laid up without notching and locking and confined with $\frac{7}{8}$ inch and 1 inch square drift bolts 20 to 30 inches long. The crown and slopes are covered with 5 inch planking. The level platforms which receive the falling water are covered with steel plates $\frac{5}{16}$ inch in thickness.

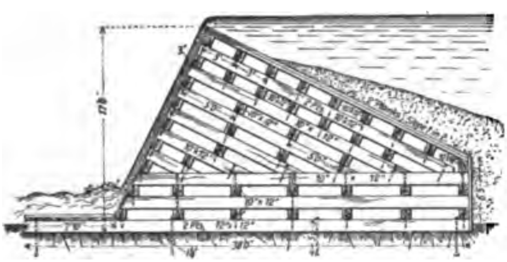


Fig. 93. Cross Section of Bear River Dam.

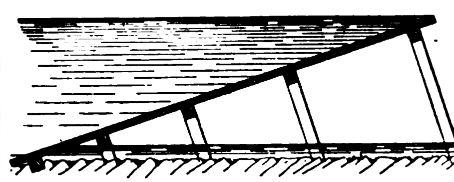


Fig. 94. Crib Dam.

Fig. 93 shows a crib dam built on the Bear River, Utah.

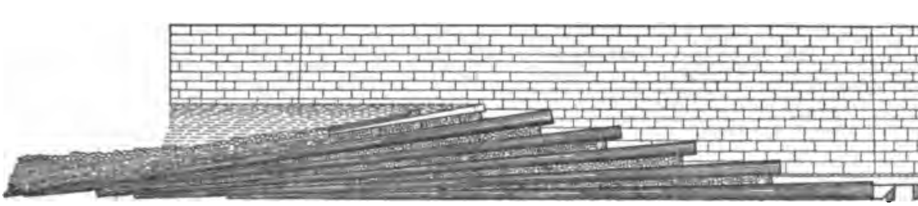


Fig. 95. The Beaver Dam.

The rafter and strut framed dam, illustrated in fig 94 is a structure the members of which may be calculated and proportioned with regard to the stresses. The members alone resist these stresses and care must be taken in the design of such dams with special regard to the possibilities

of sliding. They are commonly built with one deck, the water falling freely over the crest.

The Beaver type dam is illustrated in fig. 95, and is well adapted to small heights. This dam is of the rockfilled type, with a plank flooring extending from the crest under the earth fill which is tightly packed against the upstream side.

A series of steps is also provided to break the falling sheet of water flowing over the crest and prevent the seepage underneath.

68. Masonry Dams.

The ideal dam is unquestionably the masonry dam, built on solid rock. Such a structure inspires confidence, as it has the manifest appearance of strength and as a dam is generally intended to be a permanent construction, a masonry structure is invariably the best suited to resist the action of the elements for centuries.

Ranged ashlar or cutstone are not as suitable for masonry dams, as random rubble or rubble concrete. Stones laid in horizontal courses, would not be well adapted to resist the horizontal forces of water pressure. In all concrete work, stones for bedding should be as large as convenient handling will permit, and be placed as close together as practicable, smaller stones being imbedded in the concrete between the larger ones. In a word, the dam should be made as nearly homogeneous and monolithic as possible, interlocking the stones in all directions.

In designing a masonry dam, the following principles must be considered;

1) it must not fail by overturning;
2) the maximum compressive stresses must not exceed safe limits;
3) there must be no tensile stress at any point of the section;
4) it must resist the sliding effect of water;
5) the masonry must not be crushed;
6) it must be guarded against possible temperature cracks; and
7) also against the effect of ice expansion.

The bearing power of soils is given in Table XXXV.

Table XXXV. *Safe bearing power of soils.*

Kind of material.	Safe bearing power in tons per square foot.	
	Minimum.	Maximum.
Rock — the hardest — in thick layers in native bed.	200	—
Rock, equal to best ashlar masonry	25	30
Rock, equal to best brick masonry	15	20
Rock, equal to poor brick masonry	5	10
Clay, in thick beds, always dry	4	6
Clay, in thick beds, moderately dry	2	4
Gravel and coarse sand, well cemented	8	10
Sand, compact and well cemented	4	6
Sand, clean, dry	2	4
Quicksand, alluvial soils, etc.	0.5	1

The experience of the past dozen years has abundantly determined that particular care ought to be taken with the foundations on which a dam rests. Most dams failures have been variously due to settling, foundations washing out, slips, percolation, water seeping along the masonry and earth division, disintegration of the spillway, water flowing over the top, musk-rats, and finally in several cases excessive leakage through the foundations. It behooves the responsible engineer of any such structure to make assurances doubly sure that conditions are right before he proceeds to build the structure itself.

Mr. Hugh L. Cooper in an address before the Western Society of Engineers describes as follows the tests made before building the Keokuk dam across the Mississippi River: "The question of the sufficiency of the rock foundation has been one requiring first thought at all times. After excavations have been carried down to a stratum that has the surface appearance of being entirely satisfactory, this surface is then washed with steel brooms and water under pressure, in order that all foreign matter may be located and removed from the surface of the rock. A careful examination is then made of this rock, and if the examination does not reveal any seams or imperfections, an exploration drill is mounted on this surface, and a drill hole put down to 35 ft. These drill holes are placed longitudinally on the axis of the dam every 36 ft. After two holes have been thus driven, an air pipe is tightly connected to one hole, and 100 lb. pressure of compressed air is applied for the purpose of observing the demonstration from the neighboring open hole. In this way the presence of mud seams is determined, and if no mud is found, and there is no difficulty in maintaining the pressure in the first hole the foundations are regarded as satisfactory. The surface of the rock is observed with levels to discover whether it is manifesting any movement under the pressure above indicated, and as the 100 lb. pressure is about four times the water pressure it will be seen that the application is rather severe".

"The driving of the exploration holes is noted by a skilled observer; the speed of the drill into its reduced size as it goes down, together with a close observation of the variations in air pressure, are plotted for the purpose of discovering whether the rock is uniformly hard. This method of observing the foundations has been followed throughout the work, and in one case a mud seam 3 in. thick was found about 7 ft. down from the apparently good foundation surface, and this 7 ft. stratum was removed, together with the clay, and the work, founded on the lower level. We have put down several holes about 100 ft. deep with well drills, and one well drill near the site was driven 800 ft. for an artesian flow, and while in rock all the way no artesian water was encountered."

"Throughout the work new masonry is always let into the solid rock at least 3 ft., and after the work has been finished, tests that have been made repeatedly show an airtight seal of the concrete to the bed rock, and this in cases where a pressure of 100 lb. has been put on to the test hole and maintained for a period of six or seven hours."

It might happen that a foundation be encountered unsuitable for sustaining a solid masonry structure and in that event one might well consider a hollow structure of either steel or reinforced concrete. The likeli-

hood, however, is that bed rock will be found and the solid masonry structure commends itself as the more appropriate.

The specific gravity of rubble masonry or good concrete is usually taken at from $2\frac{1}{4}$ to $2\frac{1}{2}$, corresponding to weights per cubic foot of 140 and 156 pounds. The allowable pressure varies from 8 to 10 tons; although in the design of the Quaker bridge dam 16.6 tons was the safe pressure assumed.

It is a well known fact that after concrete has stood for some time, a gelatinous mass rises to the top. This pulpy mass called "laitance" sets slowly, has no binding properties and little or no strength. This laitance should be removed by means of wire brushes or brooms, before new concrete is deposited on the old.

Concrete dams should be built in alternate blocks and not as a continuous wall, otherwise shrinkage cracks may occur because the concrete always cracks when setting in air.

Expansion joints are provided against contraction, these having been as follows for the Keokuk dam:

At the top of the structure, in the middle line of each pier, a layer of tar paper is set on edge when the concrete is cast. The paper is elastic and acts as a cushion against which the concrete, in the 36 ft. length between paper buffers, may expand. After the paper rots out, there will remain an interval as wide as the paper is thick between each section of the top of the dam structure.

The largest mass of concrete is below, at the location of the spillways. But experience shows that concrete is so poor a conductor of heat that the center of a large mass does not fluctuate in temperature in harmony with the atmosphere, but remains of one stable temperature all the year round. Hence, it is necessary to provide for temperature expansion and contraction only at the surface and for a certain distance into the mass. In this work, the expansion joint of tar paper on edge is made at one side of each spillway in each span and from the face the width of the tar paper into the mass of concrete. The tar paper is placed on edge between the spillway section and the pier, following the curves and vertical upstream face of the shape of the spillway.

With respect to temperature stresses, Mr. Stearns says: "In a large masonry dam the interior gradually acquires the average temperature of the year and changes but little throughout the year. In the winter the outer portions of the masonry, and especially those exposed to the air, contract both vertically and horizontally, and as the temperature remains at nearly constant in the interior and does not contract, the outer portions will crack or tensile stresses will be produced in them so that these portions of the dam become ineffective for conveying compressive stresses to the foundation. This feature is an important one to include in the computations for a dam."

The sliding tendency due to the horizontal thrust of water must be looked after, and in this respect Table XXXVI will be found useful.

The ice thrust is sometimes considered, but, to what extent it affects a profile, is difficult to say, having been variously estimated at from 30,000 lbs. to 70,000 lbs. per square foot of ice 8 in. thick (in the design for the Quaker Bridge dam it was recommended that 43,000 lb. per square foot should be considered).

Table XXXVI. *Coefficient of friction of various materials.*

Material.	Coefficient.
Granite (roughly worked) on gravel and sand (wet)	0.41
Pine (sawed) on gravel and sand (wet)	0.41
Granite (roughly worked) on sand (dry)	0.65
„ „ „ „ „ (wet)	0.47
Masonry on clayed gravel	0.577
„ „ dry gravel	0.510
„ „ moist clay	0.325
Point-dressed granite (medium) on like granite	0.70
„ „ „ „ „ common brickwork . . .	0.63
„ „ „ „ „ smooth concrete	0.62
Fine cut granite (medium) on like granite	0.58
Dressed hard limestone (medium) on like limestone	0.38
„ „ „ „ „ brickwork	0.60
Beton blocks (pressed) on like beton blocks	0.66
Common bricks on common bricks	0.64
„ „ „ dressed hard limestone	0.60

The effect can be taken into account by computing the mass of masonry above high water with a margin of strength to resist ice-pressure.

The first and probably most important problem that confronts the engineer in planning the construction of a dam, is the method of taking care of the river during construction. The diversion is generally effected by means of timber cribs, or rock fill cofferdams which are to be designed to withstand overflow in case an extraordinary flood occurs during the construction period. The coffer dam built in connection with the Iowa division of the Keokuk dam, embracing 39 acres of the orginal bed of the Mississippi River, was of such tightness with regard to the bed rock that a four inch pump, running about four hours a day all kept the water out of the cofferdam. It has not been possible to locate any water coming through the cofferdam proper, which has a length of 4700 feet, and the water that ran in came from on old canal bank, finished in 1877, and a part of the present Government lock system.

The structure is described as follows[1]: „The cofferdam is composed of cribs of square timber, the cribs being 24 ft. long and 16 ft. wide, with the tops about 15 ft. above the river bed. Each crib contains 22,000 ft. B. M. of lumber. To determine the form of the rocky surface of the river bed at each crib, a raft of the size of the crib was moored in position, and a sounding pole used to get the depth along each side. The bottom portion of the crib was then built on shore, and cut to the shape shown by the soundings. This portion was then launched and floated to its site, where it was held in position by ropes, while the upper portion was built upon it. Its own weight sunk it to the bottom and when finished to the proper height it was filled with loose rock to hold it in position. When all the cribs for the cofferdam were in place, the 12 ft. openings between them were closed by stop logs on horizontal square timbers which were lowered into place and drift-bolted against the corner posts of adjacent cribs, being

[1] Engineering News. Vol. 66. No. 13. Page 358.

flush with the outside timbers of the cribs. A sheeting of vertical planks was then spiked against the cribs and the stop-logs of the outer face of the cofferdam, each plank having been driven down hard on the bed rock before it was spiked. The cribs are braced on the inside by diagonal timbers let into the bedrock."

"To make a water tight face, clay and earth were deposited outside the sheeting, being dumped from cars on a track laid along the cribs, which track was used also for the railway cars carrying the crib timbers. This fill was finally faced with loose rock dumped from cars, as a protection against the wash of waves and current. To prevent the current from scouring away the fill deposited along the east or river wall of the cofferdam, the north or up-stream wall is extended about 50 ft. beyond the river wall, and protected by a heavy pier with battered and pointed nose. This projection checks and diverts the current so that material for the fill was dumped into comparatively still water. The cross cofferdam extends beyond the riverwall in the same way, protecting the lower part of the work and preventing the diverted current from swinging back against the lower portion."

It is often found expedient to build elaborate structural forms, either timber or steel, and experience shows, that by means of steel travellers, the work of building a dam has become greatly simplified.

69. Reinforced Concrete Dams.

In order to reduce cost and because of other advantages obtained, reinforced concrete dams have come into extensive use of late, and as they are of the cellular reinforced concrete type, in some cases power houses have been accomodated within, as illustrated in fig. 96.

In the design of such structures, the buttresses are proportioned for the total pressure, and the position of the line of pressure may be varied by altering the slope of the up-stream face. The behaviour of the resultants in a concrete steel dam is shown in fig. 97.

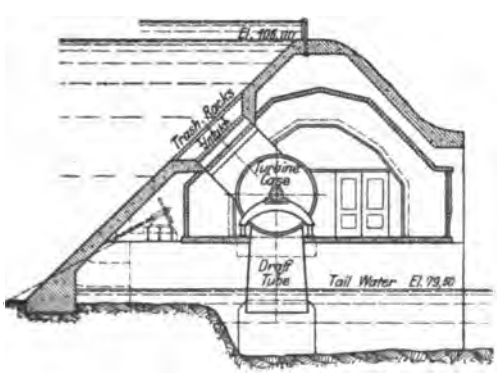


Fig. 96. Patapco Dam, Ilchester, Maryland.

A concrete steel dam consists of separate concrete buttresses supporting an inclined floor or apron of reinforced concrete. Structures of this kind require that large factors of safety be employed and to avoid internal pressures due to seepage through the up-stream face, drain openings must be provided in the down stream face, if the dam is of the spillway type.

Fig. 98 illustrates the La Prele dam, which is a 130 foot hollow reinforced concrete structure built lately across the La Prele River, Wyoming. This dam is about 330 feet long at the top and tapers down to about 110 feet at the base. Five bays at one side are provided with a spillway, normally at an elevation of 125 ft. above the floor, but the remainder of the crest is at an elevation of 130 ft. above the floor. The spillway is provided with stop logs, which raise the water levels to that of the main crest line.

Across the top of the dam, there is an 8 ft. 6 in. roadway, guarded by parapets, providing a bridge from one end to another.

A type of reinforced concrete dam has recently been proposed by Mr. Geo. J. Bancroft, which has a general cross-section similiar to that of a gravity concrete dam, but which is built of comparatively thin longitudinal and transverse walls forming vertical cells. These cells after completion are filled with earth and rock under water. For the sake of calculation however, the dam is considered as consisting essentially of a face wall supported in the rear by buttresses, which are of course, the transverse walls, that is, those running at right angles to the length of the dam. The auxiliary walls, which are parallel to the face wall and complete the enclosure of the cells, are considered only as supports, to prevent the buckling of the buttresses,

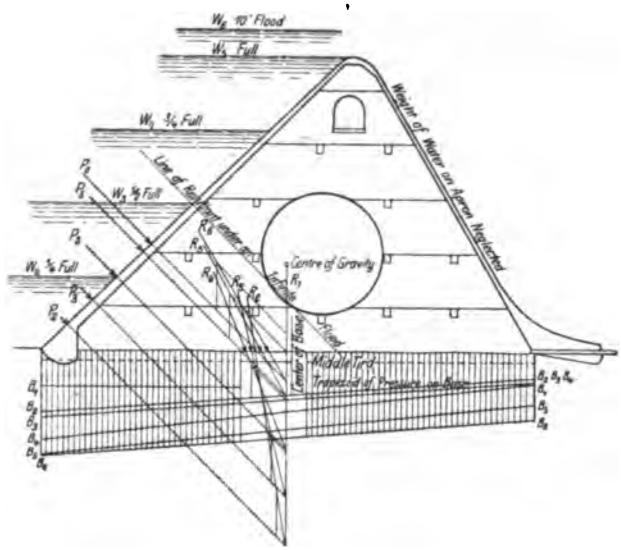


Fig. 97. Behavior of Resultants in a Concrete-Steel Dam

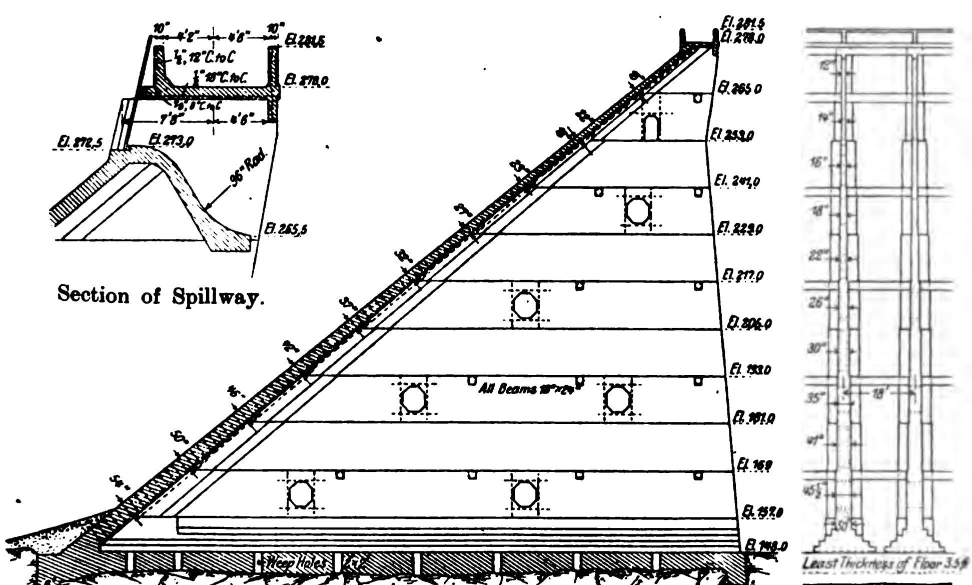


Fig. 98. La Prele Dam.

as resistance to shearing strain and as weight to resist overturning. The strength of the earth filling in shear and its supporting effect in resisting transverse and longitudinal pressure are disregarded. This type of dam,

Mr. Bancroft believes, will probably be of most use in canyon sites where the dam is a combination of gravity and arch types. The face wall, buttresses and auxiliary walls are all reinforced.

Mr. Bancroft has worked out a design of this type for a dam 185 ft. high and 1370 ft. long on the crest and arched in plan. The width of the dam on the base is 165 feet. The face wall is 2 ft. thick at the top, increasing to 7 ft. at the bottom, while the width of the entire dam at the top, in other words, the width of the front line of cells is 23 ft. $4\frac{1}{2}$ in. The buttresses and auxiliary walls are 9 in. thick at the top, gradually increasing with the depth. The buttresses are placed radially, 20 ft. apart on centers at the face wall. The auxiliary walls are 30 ft. apart on centers, except for the two nearest the face wall, which are spaced 20 and 25 ft.

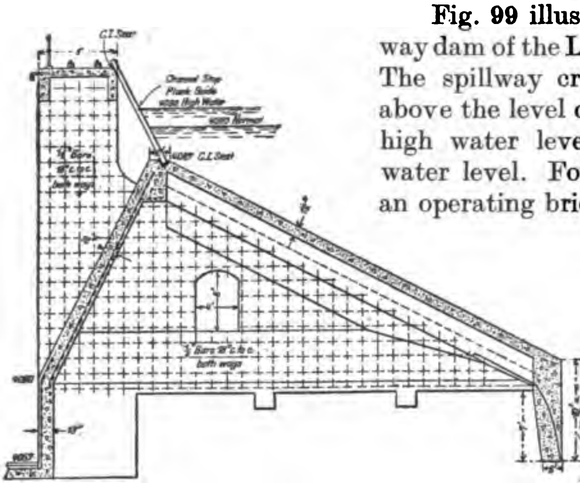


Fig. 99. Lost River Dam.

Fig. 99 illustrates a section of the spillway dam of the Lost River diversion channel[1]. The spillway crest is approximately 32 ft. above the level of the stream bed, 5 ft. below high water level, and 3 ft. above normal water level. Four feet above high water is an operating bridge, which supports one end of steel channels for carrying stop logs. The other ends of the channels rest in cast iron seats in the crest of the spillway. The level of the pond can therefore be raised by the use of stop logs to any height desired up to the level of the bridge.

The spillway as indicated in the figure i sa reinforced concrete dam with the upper and lower decks carried by concrete buttresses, which are 20 ft. on centers in the straight portion and 20 ft. on centers at the outer end in the curved portion. The downstream deck is a reinforced-concrete slab varying in thickness from 12 to 18 in., while the upper deck, on the other hand, is composed of plain concrete arches. The crown thickness is 12 in. The buttresses are 2 ft. thick, increasing to a foundation width of 5 ft. and are reinforced through the middle by $\frac{1}{2}$ in. sq. bars 18 in. center to center both ways. The buttresses rest on solid chalk rock, and to safeguard against seepage a cut-off wall is carried 7 ft. below the foundations.

70. Steel Dams.

These structures are similar in construction, to the rafter and strut-framed timber dams. They have the advantage of being cheap in construction, but are expensive in maintenance and repairs. The diagram shown in fig. 100 illustrates the frame of a steel dam built at Ash Fork, Ariz., and fig. 101 shows the details of the Hauser Lake dam, Montana. The latter is 630 ft. long and has a maximum height of 81 feet. The slope of the upstream face is 3 : 2.

[1] Engineering Record. March 18, 1911. Page 311.

One part of the foundation consists of solid rock, and the rest or about 300 feet is founded on steel sheet piles in gravel. The weight of the steel structure is 1700 tons. The lower section of the up-stream side was built of concrete, behind which rubble masonry was placed. The structural steel trusses were placed 9′ - 9″ center to center, and the up-stream face was entirely covered by steel plates extending into the concrete apron.

The Hauser Lake dam failed; the part of it which was built on soft foundation gave way.

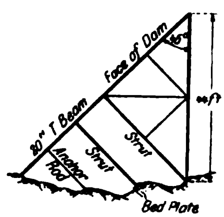


Fig. 100.
Ash Fork Steel
Frame Dam.

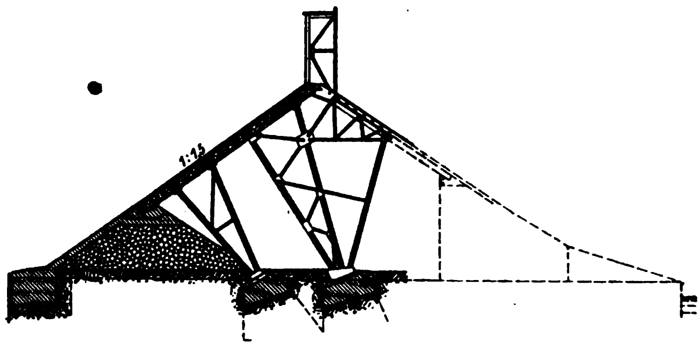


Fig. 101. Hauser Lake Steel Frame Dam.

71. The Trapezoid of Pressure.

Suppose the reservoir empty and let (fig. 102):

$G =$ normal through center of gravity;
$d =$ distance of normal to up-stream edge A;
$a =$ width of base;
$W =$ weight acting along G;

Then the average pressure along the base AB will be $\frac{W}{a}$.

The stress at A will be:

$$\frac{4a - 6d}{a} \times \frac{W}{a}.$$

The stress at B will be:

$$\frac{6d - 2a}{a} \times \frac{W}{a}.$$

The trapezoid of pressure will then be represented by MN.

If the reservoir is full, the trapezoid of pressure will be represented by $M_1 N_1$ and the average pressure will be $\frac{R}{a'}$ calling R the resultant of water and masonry pressures.

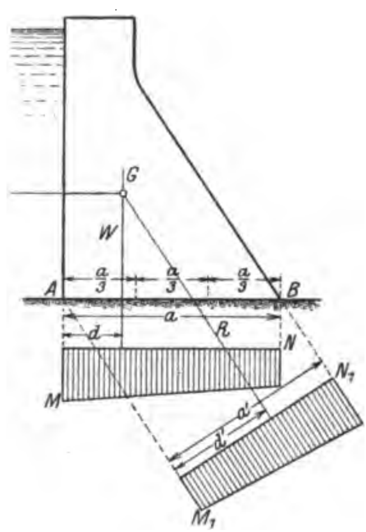


Fig. 102.

The stress at A will be

$$\frac{4a' - 6d'}{a'} \times \frac{R}{a'}.$$

The stress at B will be

$$\frac{6d'-2a'}{a'}\times\frac{R}{a'}.$$

If, at reservoir empty, the normal through the center of gravity passes through the point $\frac{a}{3}$ from A, the trapezoid of pressure is reduced to a triangle, the pressure at B being 0. If the normal falls entirely outside of the middle third, the stress will be negative, a thing which must be avoided in practice. The same reasoning applies to the resultant in case of reservoir full.

72. Stability of Low Dams.

Suppose a trapezoidal dam $ABCD$, fig. 103. Call

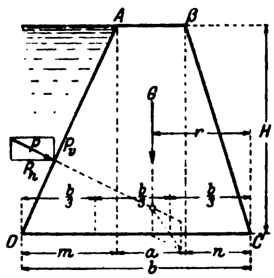


Fig. 103.

W = weight of one cubic ft. of water;
W_1 = weight of one cu. ft. of masonry;
$\frac{W_1}{W} = s$ = specific gravity of masonry;
P_v = vertical component of water pressure;
P_h = horizontal component of water pressure;

Two cases may be met with in practice: BC is vertical; or AD is vertical.

It is required to find a value for b such that the resultant of water and masonry forces intersect the base at the down stream edge of the middle third.

The vertical component of water pressure is

$$P_v = \frac{W\cdot m\cdot H}{2}$$

the horizontal component

$$P_h = \frac{W\cdot H^2}{2}$$

The force acting at G is:

$$G = W_1 H\left(\frac{a+b}{2}\right).$$

The value of r is found to be:

$$r = \frac{\frac{2}{3}n^2 + a^2 + 2an + \frac{1}{3}m^2 + am + mn}{a+b}.$$

The condition of stability is

$$P_h\times\frac{H}{3} - P_v\left(\frac{2b-m}{3}\right) - G\left(r-\frac{b}{3}\right) = 0.$$

The width of the base is therefore:

$$b = \sqrt{H^2-(a+n)^2(s-1)+n^2s+\frac{n^2s^2}{4}}-\frac{ns}{2} \quad \ldots (1)$$

If BC is vertical, $n = 0$ and

$$b = \sqrt{H^2-a^2(s-1)} \quad \ldots\ldots\ldots (2)$$

If AD is vertical, $m = 0$ and

$$b = \sqrt{\frac{5}{4}a^2 + \frac{H^2}{s}} - \frac{a}{2} \quad \ldots \ldots \ldots \ldots (3)$$

From (2), if $a = 0$, then $b = H$, and from (3)

$$b = \frac{H}{\sqrt{s}}.$$

Generally speaking it is more economical to make AD vertical rather than BC.

73. Method of Calculation of High Masonry Dams.

The method which is to be followed in the calculations for stability of high masonry dams depends entirely upon the hypothesis which is made beforehand regarding the distribution of stresses within the mass of the profile. As there is a great diversity of opinion regarding such distribution, many divergent theories, if applied to one particular problem, would lead to entirely different profiles.

It will be found interesting to study some methods which have been accepted by engineers in different countries, and when designing one particular dam, the application of Bouvier's method for instance, will be useful inasmuch as the latter is very often considered when it comes to determine the actual stresses in a masonry dam.

74. Wegmann's Method.

This consists of five stages as follows:

First stage: The depth of the rectangular portion may be determined by means of the formula (3) (indicated in paragraph 74), making

$$b = a$$

and solving for H:

$$H = a\sqrt{s}$$

in which:

H = height of rectangular portion,
a = top width,
s = specific gravity of the masonry.

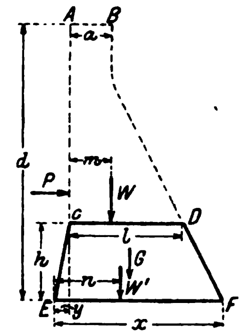


Fig. 104.

Second stage: the section below the rectangular portion is determined by considering successive trapezoidal blocks. Let $CDFE$ be such a trapezoidal block of thickness h taken very small.

This is located below the section $ABCD$, already designed.

Let, (fig. 104)

W = weight of portion $ABDC$,
G = weight of portion $CDFE$,
W' = resultant of W and G,
A = area of $ABDC$,
A' = area of $CDFE$,

$m =$ distance of line of action of W from C,
$n =$ distance of line of action of W' from E,
$l =$ length of joint CD,
$x =$ required length of joint EF,
$y =$ batter of CE,
$h =$ thickness of section,
$p =$ limiting intensity of pressure at F,
$d =$ depth of water at $E =$ heigth of dam above this point,
$q =$ limiting intensity of pressure at E, generally greater than p,
$w =$ weight of a cubic unit of water,
$w' =$ weight of a cubic unit of masonry,
$s =$ specific gravity of masonry $= \frac{w'}{w}$.

As long as $C E$ is made vertical, the value of x is:

$$x = \sqrt{B + C^2} - C \quad \ldots \ldots \ldots \ldots (1)$$

in which

$$B = \frac{d^3}{s h} + \frac{6 A m}{h} + l^2, \text{ and } C = \frac{1}{2}\left(\frac{4 A}{h} + l\right).$$

The value of n is given by

$$n = \frac{(x^2 + l x + l^2)\frac{h}{6} + A m}{A + A'} \quad \ldots \ldots \ldots (2)$$

Equation (1) can be used so long as $n > \frac{x}{3}$.

In treating the next trapezoidal section, the portion $ABFE$ is the known portion, the various properties of which are to be substituted for like properties of $ABCD$ in the above equations. Then the new value of m is n of equation (2).

Third stage. The face CE must be battered so that $n = \frac{x}{3}$, then:

$$x = \sqrt{\frac{d^3}{s h} + \left(\frac{l}{2} + \frac{A}{h}\right)^2} - \left(\frac{l}{2} + \frac{A}{h}\right) \quad \ldots \ldots \ldots (3)$$

$$y = \frac{2 A (x - 3 m) - h l^2}{6 A + h (2 l + x)} \quad \ldots \ldots \ldots \ldots (4)$$

Fourth stage. If after using formulas (3) and (4), it is found that the value of the pressure on the front face exceeds p, this formula is to be used for x:

$$x = \sqrt{\frac{w d^3}{p}} \quad \ldots \ldots \ldots \ldots (5)$$

this value being used as soon as it becomes larger than that found by means of (3). The batter is still found by (4).

Fifth stage: When the pressure on the back face equals q then

$$x = \sqrt{D + E^2} + E \quad \ldots \ldots \ldots \ldots (6)$$

in which

$$D = \frac{1}{s} \cdot \frac{d^3}{\frac{p+q}{w'} - h},$$

$$E = \frac{A + \frac{lh}{2}}{\frac{p+q}{w'} - h}.$$

The batter in this case is given by

$$y = \frac{A\,(4\,x - 6\,m) + lh\,(x - l) + x^2\left(h - \frac{q}{w'}\right)}{6\,A + h\,(2\,l + x)} \quad \ldots \ldots (7)$$

Equation (6) is to be used when the value of x is greater than that found by (5).

The value of n in this case is:

$$n = \frac{2}{3}\,x - \frac{1}{6}\,\frac{q\,x^2}{w'\,(A + A')}.$$

Although the above formulae do not take into consideration the effect of the force of waves, nor the shock produced by floating bodies, nevertheless they do give satisfactory profiles with ordinary conditions.

75. Bouvier's Method.

Consider a dam having a profile $ABCD$ as illustrated in fig. 105 and let it be required to calculate the stresses at a point M situated on the downstream side.

The effect of the resultant R on $ABMN$ is greater than that of the resultant acting on $ABME$. Effectively, in the latter case, the weight of EMN and the water pressure along EN are disregarded, both having an influence on the stress at the point M.

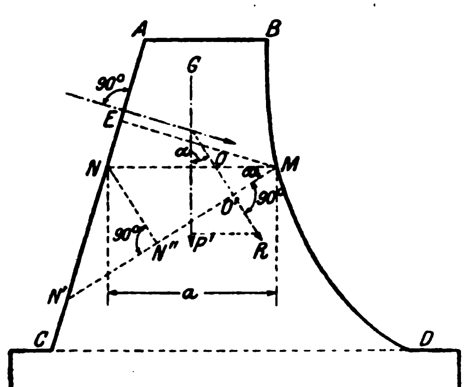


Fig. 105.

If the resultant R is distributed along MN and also along EM, it will be found that the stress in M will be greater in the first case than in the second. The result of this observation is that according to the inclination or situation of the joint, the stresses in M will be different.

In order to obtain the maximum stress in the masonry at the point M, the joint MN' which is perpendicular to the direction of the resultant ought to be considered; as it is, the effect of R on this joint is much greater as the distance $O'M$ is smaller than OM.[1]

The weight of the triangle NMN' and the effect of water pressure along NN' are neglected in computing the stress at M, and in this computation, the length MN'' is considered to be the length of the joint in consideration.

If α is the angle formed by the resultant R and the normal, a the length MN and P' the vertical component of the resultant, this resultant will have for value

$$R = \frac{P'}{\cos\alpha}$$

and according to Bouvier, will be distributed along

$$MN'' = a\cos\alpha.$$

76. Guillemain's Method.

This engineer considers the section MN', fig. 105, as resisting the weight of the section $ABMN'$, and the water pressure along AN', but not, as Bouvier, only the forces acting above MN. The result of this consideration is to give the down stream side of the dam a convex shape as shown in figure 106.

The down stream side is therefore calculated by considering each fictitions joint ab as having such a width that any oblique joint ac, ad etc., radiating about a, offers no smaller stress at a than that which is calculated by taking ab as the base.

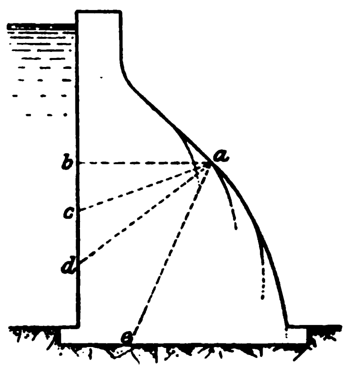


Fig. 106.

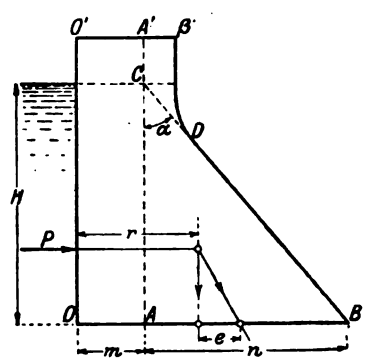


Fig. 107.

77. Levy's Method.

The principle on which this method is based consists in that at any point along the upstream face, the compressive stress in the masonry must not be less than the water pressure at the same point, and that, if any line is drawn across the profile, there be no tensile, no crushing nor sliding stresses at any point along said line.

78. Ruffieux's Method.

This French engineer gives the following method for determining the profile.

Suppose a profile as per fig. 107, and call

$$a = \text{area } O'A'OA$$
$$b = \text{area } CAB$$
$$c = \text{area } A'B'DC$$

The equation of the curve of pressures (reservoir empty) is given by

$$r = \frac{m}{2} \mp \left[\frac{m}{2} + \frac{n}{3}\right]\frac{b}{a+b} + \frac{cr_1}{(a+b+c)}$$

in which r_1 is the distance, positive or negative from the center of gravity of the area δ to the line of action of the resultant force of a and b. The equation of the curve of pressures (reservoir full) will be given by

$$e = \frac{H}{3} \frac{Q}{s(a+b+c)}$$

in which s = specific density of masonry.

If $\tan \alpha = \sqrt{\frac{1}{s}}$, the value of e becomes:

$$e = \frac{n}{3}\left(a+b+c\right).$$

It is better to make the area c as small as possible so that the point D be very little below the water level line.

79. Thiery's Formula.

Let H = height of dam;

x = length of joint at $\frac{H}{2}$;

S = slope of downstream side;

n = coefficient of resistance to compression (7 kg per sq. cm);

W = weight of one cu. meter of masonry (2250 kg)

then

$$\frac{x}{H} = \frac{-3Sn + 2\sqrt{n\left[3n\left(S^2 + \frac{1000}{W}\right) - H\left(S^2 + 4000\right)\right]}}{2(3n - 4WH)}.$$

If $S = 0.50$, the formula gives

$$\frac{x}{H} = \frac{1}{3}.$$

The width at the top will then be

$$b = \frac{H}{2}\left(\frac{2}{3} - \frac{1}{2}\right)$$

and the width at the base:

$$B = \frac{H}{2}\left(\frac{2}{3} + \frac{1}{2}\right).$$

80. Stability of Masonry Dams. Example of Calculation.[1]

Suppose a section of masonry dam as per fig. 108. The weight of masonry will be taken at 140 lbs. per cu. ft. The weight of water = 62.5 lbs. per cu. ft.

The water pressure at any point is given by

$$P = \frac{62.5\,H^2}{2}.$$

The dam being 30 feet high, 6 divisions of 5 feet each are made. Call W_1, W_2, W_3 etc., the weight of the masonry above joint 1, 2, 3 etc.,

[1] All calculations have been made with a slide rule.

and R_1, R_2, R_3 the resultant forces of water pressure and weight of masonry at the subdivisions.

Denoting by A_1, A_2, A_3 etc., the area above each joint, we have

$$A_1 = 5 \times 5 = 25 \text{ sq. ft.} \qquad W_1 = 25 \times 140 = 3500 \text{ lbs.}$$

$$A_2 = A_1 + \frac{8+5}{2} \times 5 = 57.5 \text{ sq. ft.} \qquad W_2 = 57.5 \times 140 = 7550 \text{ ,,}$$

$$A_3 = A_1 + A_2 + \frac{8+11}{2} \times 5 = 105 \text{ sq. ft.} \qquad W_3 = 105 \times 140 = 14700 \text{ ,,}$$

$$A_4 = A_1 + A_2 + A_3 + \frac{11+14}{2} \times 5 = 167.5 \text{ sq. ft.} \qquad W_4 = 167.5 \times 140 = 23450 \text{ ,,}$$

$$A_5 = A_1 + A_2 + A_3 + A_4 + \frac{14+17}{2} \times 5 = 245 \text{ sq. ft.} \qquad W_5 = 245 \times 140 = 34400 \text{ ,,}$$

$$A_6 = A_1 + A_2 + A_3 + A_4 + A_5 + \frac{17+20}{2} \times 5 = 337.5 \text{ sq. ft.} \qquad W_6 = 337.5 \times 140 = 47250 \text{ ,,}$$

The water pressures are as follows

$$P_1 = \frac{62.5 \times 5^2}{2} = 382 \text{ lbs} \qquad P_2 = \frac{62.5 \times \overline{10}^2}{2} = 3125 \text{ lbs}$$

$$P_3 = \frac{62.5 \times \overline{15}^2}{2} = 7050 \text{ lbs}$$

$$P_4 = \frac{62.5 \times \overline{20}^2}{2} = 12500 \text{ lbs}$$

$$P_5 = \frac{62.5 \times \overline{25}^2}{2} = 19500 \text{ lbs}$$

$$P_6 = \frac{62.5 \times \overline{30}^2}{2} = 28125 \text{ lbs.}$$

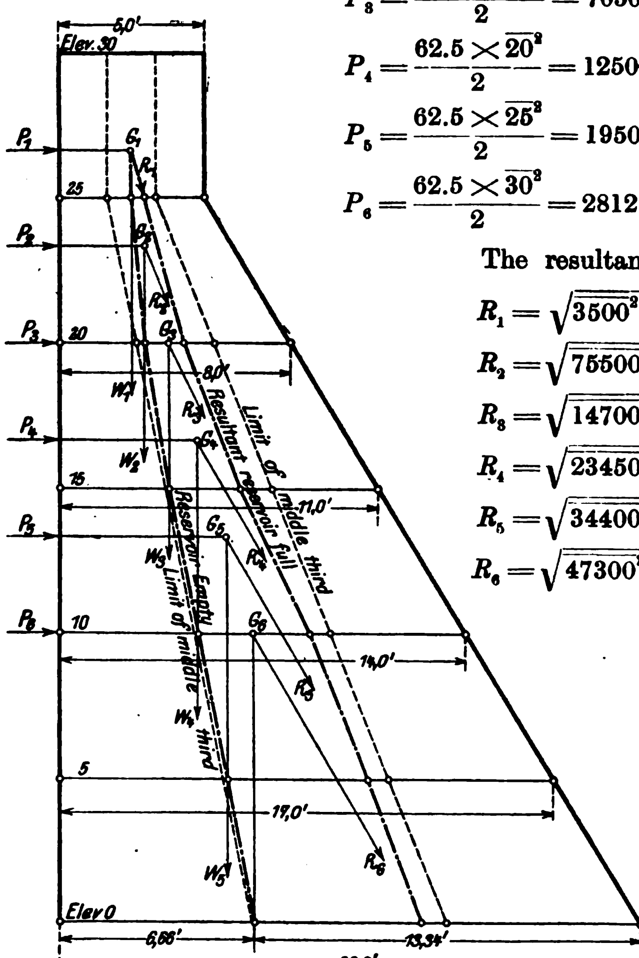


Fig. 108.

The resultants will have for value

$$R_1 = \sqrt{\overline{3500}^2 + \overline{382}^2} = 3520 \text{ lbs}$$

$$R_2 = \sqrt{\overline{75500}^2 + \overline{3125}^2} = 8100 \text{ lbs}$$

$$R_3 = \sqrt{\overline{14700}^2 + \overline{7050}^2} = 16300 \text{ lbs}$$

$$R_4 = \sqrt{\overline{23450}^2 + \overline{12500}^2} = 26600 \text{ lbs}$$

$$R_5 = \sqrt{\overline{34400}^2 + \overline{19500}^2} = 39500 \text{ lbs}$$

$$R_6 = \sqrt{\overline{47300}^2 + \overline{28125}^2} = 55000 \text{lb s}$$

From figure 108, it is seen that the resultant lines both for reservoir full and reservoir empty are inside the middle third.

The overturning moment of water is

$$P_6 \times 10 = 28125 \times 10 = 281\,250 \text{ ft. lbs.}$$

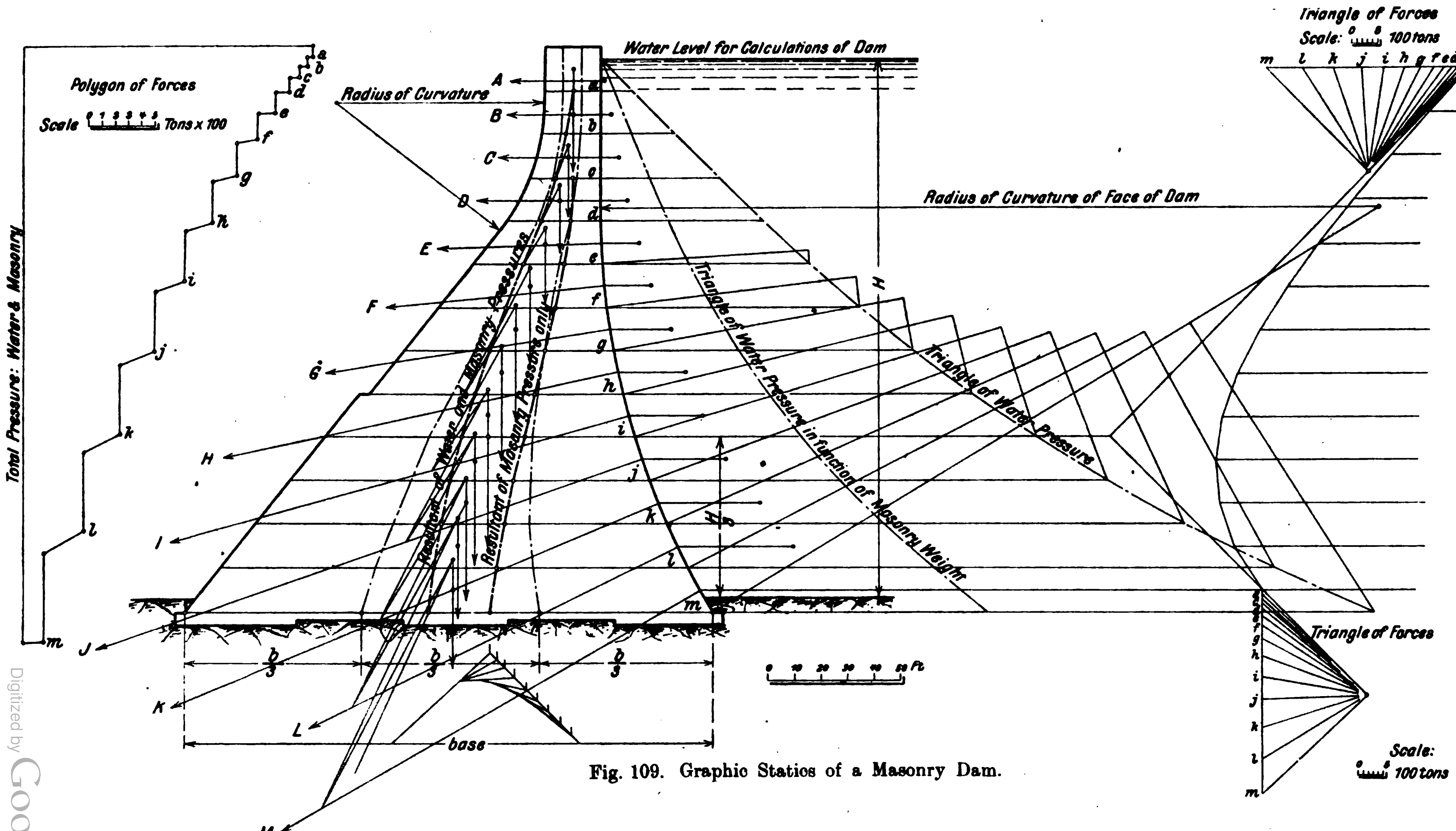


Fig. 109. Graphic Statics of a Masonry Dam.

The resisting moment of the masonry is

$$W_6 \times 13.34 = 47250 \times 13.34 = 630\,315 \text{ ft. lbs.}$$

The factor of safety against overturning is therefore:

$$\frac{630\,315}{281\,250} = 2.25 .$$

For higher dams, the compressive strength of the masonry would have to be considered, but for the case here studied the above analysis is quite sufficient.

81. Graphical Analysis of Masonry Dams.

The investigation of stability of dams by means of graphic statics is illustrated in fig. 109, which is self-explanatory. As in the analytical method, the cross section must be subdivided into trapezoids, and the point where the resultant cuts the successive joints is determined by means of the triangles of forces as illustrated.

82. Upward Pressure due to Seepage.

Percolation exists in and beneath most masonry dams, and the result of this percolation is the development of uplift pressures. In cases of serious infiltration, a large portion of the weight of the structure is lifted and the sliding resistance is thereby greatly reduced. There is no doubt that water under pressure can penetrate the horizontal joint between the masonry and the bed rock, and also the planes joining the days work. A failure of a dam in such conditions is to be feared.

In order to prevent infiltration and therefore upward lift, Mr. F. R. Hayton suggests the building of a trench in the rock parallel to the up-stream side of the dam and spaced 5 to 10 feet in front of it. Such a trench may be 2 or 3 feet in depth, and is to be filled with coarse gravel or rock so that water can readily go through it. At the center or lowest point, a drain pipe is run under the dam to a point below it for the purpose of conducting any leakage into the trench outside. Such a pipe must be well grouted with a rich cement to ensure no leakage below the trench or weakness in the structure.

It has also been thought that another solution of this problem consisted in anchoring the dam down to the rock by means of rails and chains, and depending on such anchorage to resist sliding, which as has been said, may be caused by infiltration of the water under pressure between the foundation and the bottom of the dam. Such anchoring is dangerous because it is difficult to anchor a steel rail into the rock for such purposes and the steel itself may rust on account of masonry dampness.

The best method seems to be the arching of the dam in plan and solution by this means is to be sought whenever possible.

83. Curved Dams.

A wide difference of opinion still exists among engineers as to the effective value of the arch in dam construction.

According to Delocre, a curved dam will act as an arch if its thickness does not exceed one-third of the radius of its upper face, while J. B. Krantz asserts that a radius as small as 65 ft. is ample to allow a dam to act as an arch and transmit the water pressure to the sides. It is undoubtedly true that such structures do transmit the water pressure laterally, and the best proof of this statement consists in the fact that some dams of this type are so slender in profile as to be absolutely unstable were they built straight.

It is a recognized fact that the curving of a dam enables it to resist the tendency to vertical cracks due to temperature variations.

Mr. H. M. Wilson[1] says: "An additional advantage of the arched form of dam is that the pressure of the water on the back of the arch is perpendicular to the up-stream face, and is decomposed into two components, one perpendicular to the span of the arch and the other parallel to it. The first is resisted by the gravity and arch stability, and the second thrusts the up-stream face into compression, which has a tendency to close all vertical cracks and to consolidate the masonry transversely. An excellent manner in which to increase the efficiency of the arch action in a curved dam is that employed in the Sweetwater dam. This consists in reducing the radius of curvature from the center towards the abutments. The good effect of this is to widen the base or spring of the arch at the abutments, thus giving a broader bearing for the arch on the hillsides. The effect of this is seen in projections or rectangular offsets made on the down stream face of the dam, the center sloping evenly, while the surface is brocken by steps when it abuts against the hillsides . . . . Though the cross-section of a curved dam may unquestionably be somewhat reduced, it would be unsafe to reduce it as much as has been done in the case of the Bear Valley and Zola dams, though these have withstood securely the pressures brought against them. It might with safety be reduced to the dimensions of the Sweetwater dam, thus saving largely the amount of material employed".

In England there are several dams which have bases equal to about one third their heights, and in one instance this ratio is as small as one-seventh. In the design of such dams, the following formula is used:

$$T = \frac{RP}{S}$$

in which

T = thickness of the arch;
R = radius of up-stream face;
P = water pressure;
S = allowable unit compression stress in the arch.

This formula naturally assumes a shell so thin as compared with the radius that the compressive stress is uniformly distributed over the

[1] H. M. Wilson. Manual of Irrigation Engineering.

cross-section. When the thickness becomes large in comparison with the radius, this assumption should be no longer made, for the stress is not then uniformly distributed. In such a case the dam really becomes a thick, hollow cylinder and should be designed accordingly.

Some engineers are of the opinion that when the upper part of a curved dam is of greater length than the lower part, there is a greater deformation at the top, and undeterminate stresses are introduced rendering calculations very difficult. It is now commonly conceived that the curved dam is best suited to a deep and narrow gorge, where these unequal stresses are unlikely to be of any importance.

An arched dam of a type in many respects different from ordinary American practice has been designed for the Agua Pura Company, of Las Vegas, N. M. This dam, fig. 110, will have a maximum height of 50 ft. and a thickness of 15½ ft. at the base, corresponding to a pressure in the arch of 350 lbs. to the square inch. It has been designed with the idea of later increasing its height.

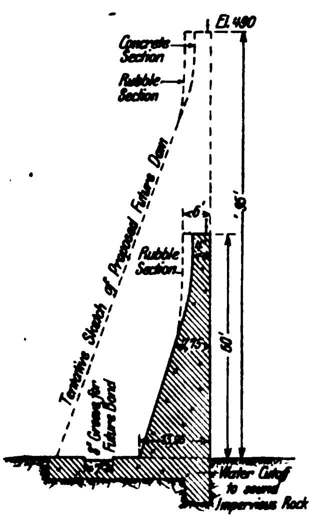


Fig. 110. Section through Crown of present 50-Ft. and proposed 95-Ft. Las Vegas Arched Dam.

As between an arched dam and a straight dam with a gravity section, estimates showed that the former type would require less than two thirds as much masonry as the latter. The mouth of the canyon is narrow, with rather steep sides and with rock showing at the surface over about one half of the site of the dam. The contour of the surface is well suited for the abutments of an arched dam. Several trials led to the laying out of an arched dam with a radius of 250 ft. for the up-stream face, in which the curve would be approximately normal to the direction of the contours. Designs were therefore completed on this basis.

Table XXXVII, compiled by Mr. C. W. Sherman, gives the principal details of curved masonry dams. A reinforced masonry dam of the arch type built on the Cache la Poudre River, Colo., is analyzed as follows by its designer, G. N. Houston in the Engineering Record: the analysis of the stresses in the dam being based on the following assumptions:

a) That the high water level is 9.7 ft. above the spillway, that is, to the top of the dam.
b) That the dam resists the water pressure partly as a cantilever beam and partly as an arch.
c) That the center of gravity of the compressive forces is at a point distant from the down stream face 0.15 of the effective thickness of the dam.
d) That the dam section is a complete triangle, thus making the perpendicular line through the center of gravity of the section cut the base at a point one third the thickness of the dam from the rear face.
e) That the safe load (tension) on the reinforcement is 12 000 lb. per sq. in.

f) That the neutral axis of the dam, considered as a beam, is at a point 0.4 the effective thickness from the down stream face.

g) That the thrust in the arch is uniformly distributed over the thickness of the dam.

Table XXXVII. *Principal details of curved masonry dams.*

Place	Max. Ht. Feet	Thickness Base	Thickness Top	Max. Stress in Arch. Lb. per Sq. In.	Rad. Up-Stream Face	Length Top	Character of Rock	Date
Katoomba, N. S. W. . . .	25	20.29	3.0	233	220	320	Sandstone	1905
Picton, N. S. W.	28	13.62	7.0	186	120	112	Sandstone	1897
Winchester, Ky.	31	8.58 at 15 ft. above base	4.83	498	318.4	407		
Qn. Charlotte Vale, N. S. W.	32	8.65	3.0	155	90	113	Quartzite	1898
Parkes, N. S. W.	33.5	13.5	3.0	373	300	540	Granite	1897
Lithgow No. 1, N. S. W. .	35	10.88	3.5	155	100	178	Sandstone	1896
Wollongong, N. S. W. . .	42	11.62	3.5	311	200	535	Basalt	—
Cootamundra, N. S. W. .	46	13.0	3.0	389	250	640	Granite	1898
Wellington, N. S. W. . . .	48	10.0	3.0	311	150	350	Conglomerate	1899
Bear Valley, Cal.	64	8.4 at 48 ft.	3.17	825	335	300	Granite	—
Mudgee, N. S. W.	50	18.0	3.0	311	253	498	Altered slate	1899
Parramatta, N. S. W. . .	52	15.0	4.8	223	160	225	Sandstone	—
Lewiston, Ida.[1]	55.5	14.5	5.33	475	286.5	288		
Tamworth, N. S. W. . . .	61	21.5	3.0	311	250	440	Granite	1898
Medlow, N. S. W.	65	8.96	3.5	186	60	124	Sandstone	1906
Lithgow No. 2, N. S. W. .	87	24	3.0	155	100	221	Sandstone	1906
Sweetwater, Cal.	90	46	12	188	222	380		
Ithaca, N. Y.	90	7.75	—	283	67.85	—	Shale	1903
Barossa, S. Aus.	95	34	4.5	242	200	—	—	1903
Zola, France	119.7	41.82	19.02	196	158	205		
Cheesman, Col.	210 to H. W.	176 72.66 at	18 E 1.100	— —	399.89 at top	—	Granite	1900
Pathfinder, Wyo.	210	94	10	181	(150 at center) (186.5 at base)			
Las Vegas	50 95	15.50 43.30	5.0 5.0	350 300	250 250	210 390	— Sandstone	1910 —

The general method used is to assume the safe load on the reinforcement, find what part of the water pressure can be safely carried by the dam considered as a beam, and then assume that the arch carries the remainder of the pressure.

In the following equations the various functions are represented by the following letters:

$P =$ total overturning pressure on the dam, in tons;

$P'' =$ that part of P assumed to be sustained by the beam action, in tons;

$P' =$ that part of P assumed to be sustained by the arch action in tons;

$w =$ weight of 1 cu. ft. of water in lbs.;

$r =$ radius of curvature of dam in feet, $= 324$;

$t =$ thickness of the dam at any point, in feet;

$h =$ height of the dam at any point in feet;

W = weight of masonry in the dam above any point, in tons;
T = thrust in arch due to P', in tons per square foot;
y = assumed stress in steel, in tons;
c = maximum compression in the outer face of the dam due to beam action alone, in tons per square foot;

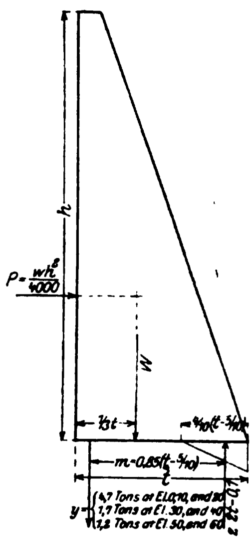


Fig. 111. Diagram of Stresses acting of the Dam.

z = total compressive stress in dam due to beam action alone in tons;
q = that part of the pressure at any point sustained by arch action alone, in tons.

A diagram of the stresses acting on the dam is given in Fig. 111.

Expressing moments around the center of gravity of compression forces, we get

$$\frac{1}{3}P''h - W\left(m + 0.5 - \frac{1}{3}t\right) - ym = 0.$$

Solve for P''.

Then

$$P' = P - P''$$

$$q = \frac{2P'}{h}$$

$$T = \frac{qr}{t}.$$

To determine c, take moments around y:

$$\frac{1}{3}P''h + W\left(\frac{1}{3}t - 0.5\right) - c\left(0.2\,t - 0.1\right)m = 0.$$

Solve for c.

The stresses obtained by this method are shown in Table XXXVIII.

Table XXXVIII. *Summary of stresses.*

Elevation.	Tension in steel, in tons per square inch.	T	c
62.3 Spillway	0.0	0.0	—
60	0.0	0.0	—
50 Calculated	4.5	0.0	5.8
40 Assumed	6.0	4.4	8.0
30 "	6.0	7.2	9.7
20 "	6.0	7.5	12.3
10 "	6.0	9.3	13.5
0 "	6.0	10.5	15.7
0 "	10.0	9.1	16.3

The factor of safety is about 6.

84. The Spillway Dam.

This sort of dam is frequently used today in connection with low and medium head developments on rivers with a large discharge.

The author[1] has determined an equation of the economical minimum shape for a spillway of the ogee type assuming the following data:

[1] La Technique Moderne. Paris, Aug. 15 and Sep. 1st 1912.

Weight of one cubic foot of water = 62.5 lbs.
Density of masonry = 2.5
The equation is

$$B = \frac{3\sqrt{hx}[x + H] + hx}{4x}$$

where (see fig. 112).

B = width of base (toe not considered);
H = height of spillway in feet;
h = height of overflow in feet;
x, y = coordinates of parabolic face of dam.

Table XXXIX gives the values of x and y for dams from 20 to 100 feet high and overflows from 2 to 16 feet deep.

For any case, knowing H and h take the corresponding value of x as per table and substitute in above equation.

Suppose a spillway dam to be 70 feet high with an overflow 9 feet deep over the crest: The corresponding value of x is 21.16, from table XXXIX, which substituted in the above equation gives:

$$B = \frac{[9\sqrt{21.16} \times 91.16] + [9 \times 21.16]}{4 \times 21.16}$$

$B = 46.84$ feet.

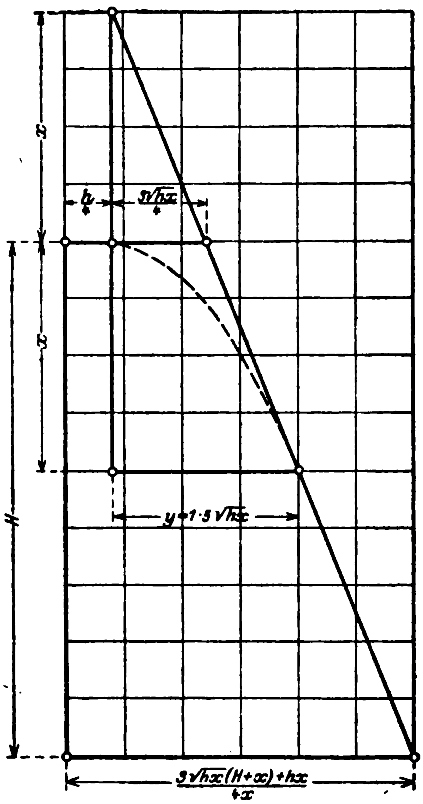


Fig. 112.

Of the several elements which enter into the computations of stability of a spillway dam these only enter in this equation

1st: Water pressure for static conditions, weight of the overflowing nappe not being considered;

2nd: Weight of masonry acting by gravity alone;

3rd: The possibility of vacuum effects under the overflowing sheet is eliminated, the parabolic face the dam cutting into the falling sheet.

Having therefore determined the width of the base for a specified case, it is obvious that there still remains to analyze the profile with reference to the other conditions which render a dam safe and which are enumerated in the paragraph on masonry dams.

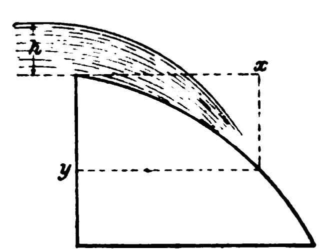


Fig. 113.

Another formula giving the depth of the joint in which the resultant of water and masonry forces leaves the middle third is the following[1]:

$$y^2 - 3.8\,hy - 12.8\,h^2 = 0$$

[1] Atti della Associazione Elettrotecnica Italiana. Vol XV. Page 757.

in which

h and y are as per figure 113, being given the condition

$$y = \frac{x^2}{4h}.$$

The figure shows, however, that the ogee face extends down to the base of the dam, the profile being therefore different to that shown in fig. 112.

Table XXXIX.

Height of Spillway	Height of Overflow h							
	$h=2$		$h=4$		$h=9$		$h=16$	
H	y	x	y	x	y	x	y	x
20	2.24	4.00						
25	2.15	3.65	3.98	13.25				
30	2.11	3.50	3.51	10.04				
40	2.06	3.31	3.20	8.18				
50	2.03	3.20	3.08	7.51	6.79	39.44		
60	2.01	3.13	3.03	7.13	5.44	24.30		
70	2.00	3.10	2.96	6.86	5.11	21.16		
80	1.99	3.06	2.91	6.60	4.89	19.18		
90	1.98	3.03	2.87	6.40	4.77	18.15	8.98	68.98
100	1.97	2.99	2.85	6.30	4.67	17.31	7.77	50.27

85. The Siphon Spillway.

Ordinarily, the surplus of a river flow is discharged over a spillway dam of sufficient length to let the required amount pass. When however, the volume is large, the spillway must be long, and sometimes conditions exist which make a long spillway undesirable or impossible.

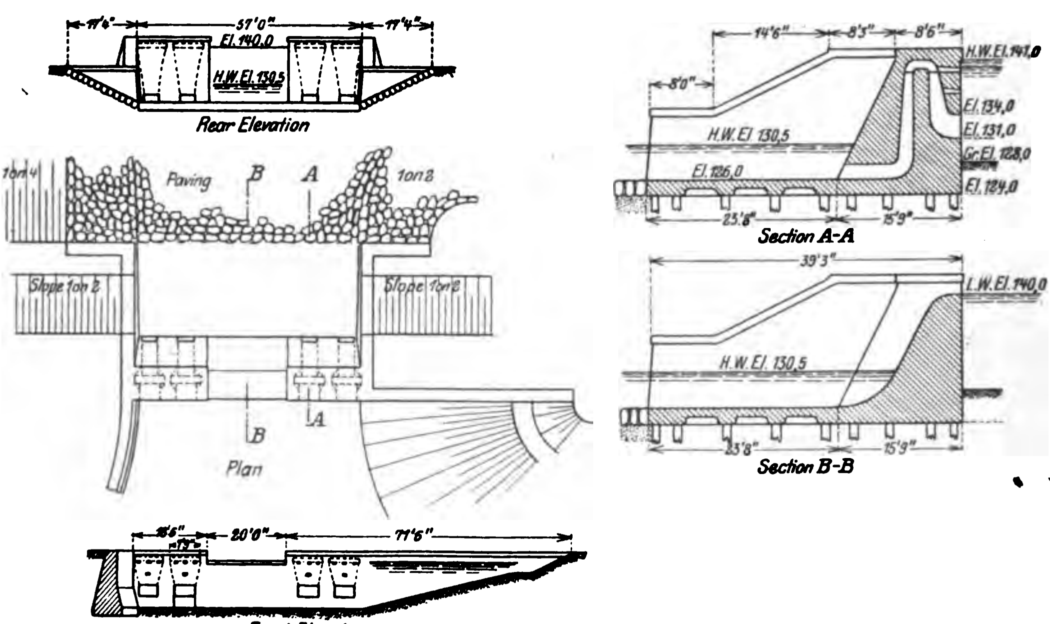


Fig. 114/115. Details of Siphon Spillway at Lock No. 9. Champlain Canal.

Fig. 114/115 shows a section of siphon spillway used on the Champlain Canal. The siphon action is automatic in both starting and stopping of the flow, and the fluctuation of the water surface is limited to about one foot. Each of the siphons has an area of $7\frac{3}{4}$ sq. ft. Acting under a head of 10.5 ft., each will discharge about 160 cu. ft. per sec. The inlet of the siphon is placed well below the water surface, and is protected by a screen to prevent the entrance of floating bodies that might happen to drift into the pipe and lodge in one of the several bends of the siphon. In order to reduce the loss of head due to entry, the inlet is flared out to double the normal area. As the siphon will not come into action until completely filled with water, it was necessary to limit the height at the crown to 1 ft., but the necessary area was obtained at this point by increasing the width. From the crown to the outlet the siphon is practically uniform in area, though the dimension were changed from $1 \times 7\frac{3}{4}$ ft. to 2×4 ft., in order to give a better section to the masonry, and also to facilitate the removal of forms used in construction. At the crown, where the removal of forms would have been difficult, iron castings were provided, which were left in the masonry. Three vents, each 6 in. high, 12 in. long, pierce the wall at low water level above the inlet of each siphon. When the water has been drawn down to this level, air will enter through these vents and stop the flow through the siphon. A little below these vents a single opening of the same size acts as a precautionary vent to break the flow in case the upper openings become clogged in any way, by freezing or otherwise.

86. Flashboards. Mechanical Aspects.[1]

In order to increase the height of a dam during low water and so to dispose of a higher head than would be otherwise possible at the moment when such increase is most opportune, it frequently is necessary to adopt a system of flashboards designed to give way when the water reaches a specified elevation. In the case where iron pins are used to resist the water pressure on the flashboards, a certain fiber stress per unit area is assumed, at which the pins are expected to bend and release the boards. However it must not be forgotten that some variation naturally occurs in the condition of the fiber of the iron used for such pins, and in assuming a definite maximum fiber stress as a base for calculations, such stress may vary in such a large proportion to the actual existing stress when the pins begin to yield, that probably the system as calculated will not give the results expected and the job of calibrating the pins or adjusting the height of the flashboards will be left to actual experience after a few seasons of actual service.

This study consists in the development of a formula which shall give off hand and as accurately for practical purposes as the variation in fiber will permit, the maximum head of water over the crest of the dam for which the pins will bend and release the boards. With this end in view, an analysis will first be made of the bending conditions of the pins from actual tests. The calculations are based on the results of observations of the flashboards at the Essex dam, Lawrence, Mass., see fig. 116.

[1] The Author in Engineering Record. Aug. 22. 1908.

The pins are of Wayne iron, $1\frac{1}{2}$ in. diameter and 3 ft. 6 in. long. These pins are placed in holes in the masonry about 6 in. in depth and 20 in. apart. The flashboards are 3 ft. high, reaching the top of the pins when in position. The crest of the dam is at El. 34.12, the top of the flashboards at El. 37.12. When the water reaches El. 40.20, the pins begin to bend.

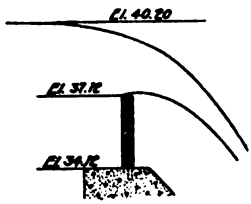


Fig. 116.

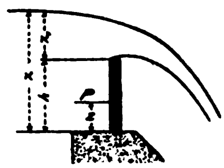


Fig. 117.

In the following study (see fig. 117) let

x = height of water in feet above dam crest when pins begin to bend;

x_1 = height of water in feet above flashboards when pins begin to bend;

$x - x_1 = h$ = height of flashboards in feet;

s = spacing of pins in feet;

d = diameter of pins in inches.

The total pressure pertaining to one pin is

$$\frac{62.5\,(x - x_1)\,(x + x_1)}{2}.$$

The water pressure on an element dx of the flashboards is $62.5\ sx\,dx$. The moment of such an element with reference to the surface is $62.5\ sx^2\,dx$.

The total moment is

$$62.5\,s\int_{x_1}^{x} x^2\,dx = \frac{62.5\,s\,(x^3 - x_1^{\,3})}{3}.$$

Call z_1, the depth of the center pressure, then

$$62.5\,s\,(x^3 - x_1^{\,3}) \times \frac{1}{3} = 62.5 \times \frac{1}{2} \times s\,(x^2 - x_1^{\,2})\,z_1$$

or

$$z_1 = \frac{62.5\,s\,(x^3 - x_1^{\,3}) \times 2}{62.5\,s\,(x^2 - x_1^{\,2}) \times 3} = \frac{2\,(x^3 - x_1^{\,3})}{3\,(x^2 - x_1^{\,2})}.$$

Therefore, the arm of the bending moment will be

$$z = x - z_1 = x - \frac{2\,(x^3 - x_1^{\,3})}{3\,(x^2 - x_1^{\,2})}.$$

The bending moment itself may now be written

$$M = 62.5\,\frac{s}{2}\,(x^2 - x_1^{\,2})\left[x - \frac{2\,(x^3 - x_1^{\,3})}{3\,(x^2 - x_1^{\,2})}\right] \times 12$$

$$= 375\,s\,(x^2 - x_1^{\,2})\left[x - \frac{2\,(x^2 - x_1^{\,3})}{3\,(x^2 - x_1^{\,2})}\right] \quad . \ . \ . \ . \ . \ . \ . \ . \ (1)$$

Let us apply this formula to find the moment for the Lawrence River dam flashboards:

Here

$$x = 40.20 - 34.12 = 6.08,$$
$$h = 3,$$
$$x_1 = 6.03 - 3 = 3.08,$$
$$s = 1.667 \text{ ft.},$$
$$d = 1\tfrac{1}{2} \text{ inch.}$$

Substituting respectively these values in (1) we find:

$$M = 22{,}800 \text{ inch-pounds.}$$

This moment is equal to the resisting moment of the pins, which is

$$\frac{\pi f d^3}{32}.$$

Therefore

$$0.098\, f d^3 = 22{,}800 \text{ inch pounds;}$$
$$f = 69{,}500 \text{ lbs per square inch..}$$

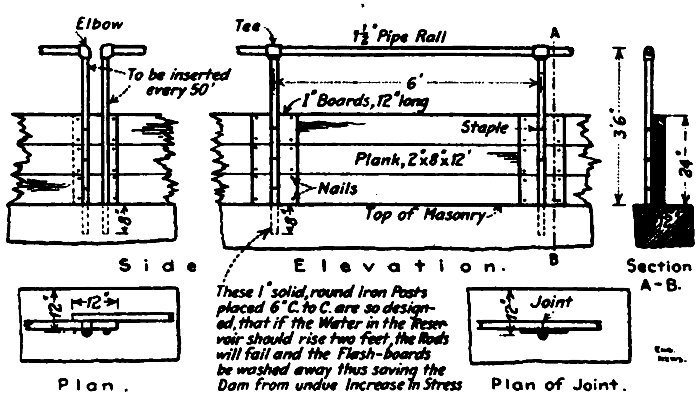


Fig. 118. Detail of Flashboard Construction.

It is seen then that when the pins begin to bend, the extreme fiber stress is 69,500 lb. per square inch.

Assuming now this amount of fiber stress per unit area, we will develop a formula in which, by interpolation, we will be able to establish the different functions exerting their influence on the bending of the pins.

Returning to expression (1) and equating it to the resisting moment of the pins, we have for a general case:

$$375\, s\,(x^2 - x_1^2)\left[x - \frac{2}{3}\frac{(x^3 - x_1^3)}{(x^2 - x_1^2)}\right] = 0.098\, f d^3,$$

f being 69,500, as determined before, so

$$s\,(x^2 - x_1^2)\left[x - \frac{2}{3}\frac{(x^3 - x_1^3)}{(x^2 - x_1^2)}\right] = 18.12\, d^3.$$

Decomposing in factors and remembering that $x - x_1 = h$ we have

$$sh\,(x^2 + x x_1 - 2 x_1^2) = 54.36\, d^3.$$

But from $x - x_1 = h$, there results $x_1 = x - h$.

Substituting this value for x_1, we derive

$$sh\,(3\,xh - 2\,h^2) = 54.36\,d^3$$

from which

$$x = \frac{18.12\,d^3}{sh^2} + \frac{2}{3}h.$$

This formula gives theoretically the head of water that will make Wayne iron pins bend, knowing the diameter and spacing of pins and the height of flashboards.

To check the formula with the Lawrence dam flash boards make

$$d = 1\tfrac{1}{2},$$
$$s = 1.667,$$
$$h = 3,$$

we have

$$x = \frac{18.12 \times 3.375}{1.667 \times 9} + 2 = 4.08 + 2 = 6.08.$$

It seems manifestly fair, then, to apply this formula to all cases where the same kind of iron as here is used for the pins.

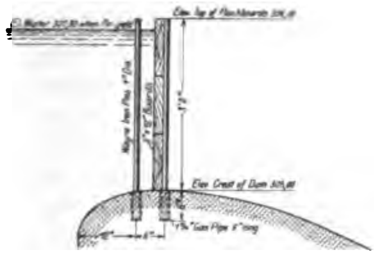
Fig. 118, shows the details of flashboard construction in connection with the Las Vegas dam, and fig. 119 gives the flashboard details of the Schaghticoke dam.

87. Movable Dams.

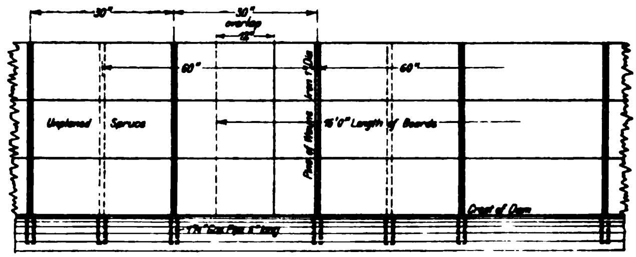
These are usually used when the low water surface can be raised to an appreciable extent, provided flood conditions are taken care of. Movable dams are so constructed as to be raised or lowered according to the fluctuations of the water level. The moving parts may consist of a gate, such as the Stoney Roller sluice gate, Fig. 120. These gates move in vertical grooves on roller trains in the abutting piers. The arrangement is such that the gates move twice as fast as the rollers; both the rollers and the gates being counterbalanced, it is easy to operate them by hand.

Another movable dam is illustrated in Fig. 121, built in connection with the Rio Guaso irrigation system. When in service, the shutters are maintained in a vertical position by the pressure of the water on the upstream side forcing the lower edge against a stop in the top of the spillway, and pulling against inclined tension rods pivoted at the upper end to the shutters near their center point and anchored to the masonry at their lower end. The tension rods are adjustable by turnbuckles and can be connected to the shutters at six different points to provide for corresponding heights of water. When the water rises on the upstream side to a certain height above the pivoted connections, the unabalanced pressure on the shutter causes the upper edge to revolve downward and the shutter assumes a horizontal position, allowing the flood water to pass without much obstruction. After the flood has passed, the shutters are easily lifted by two men, with a chain hoist suspended from a light movable frame.

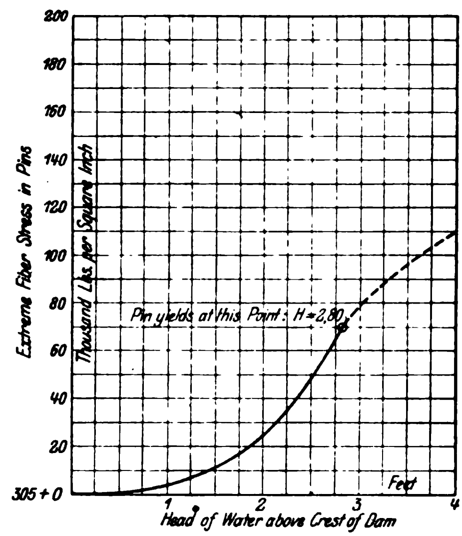
When they are collapsed the lower edges of the shutters, fitted with solid curved steel shoes, travel horizontally on steel track plates. Blowoff pipes, closed at the upper ends by conical valves, are provided to take the sediment out of the masonry recess at the foot of the shutter gates.



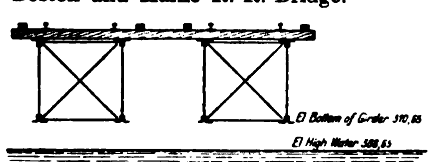
Transversal Elevation of Flashboards.

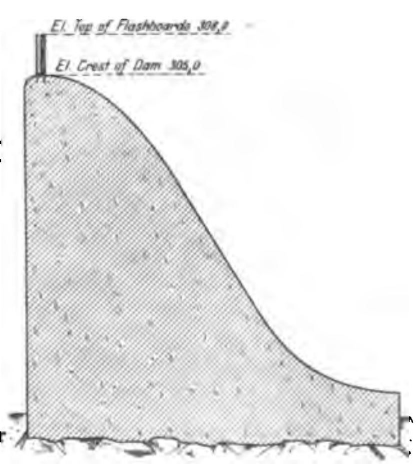


Longitudinal Elevation of Flashboards.



Boston and Maine R. R. Bridge.





Max. high water elevation over Grist Mill dam . . 301.00
Elev. crest of dam 295.00
Head 6.00 Ft.

Length of old dam 390 Ft.

$$Q = 3.33 \times 390 \times (6)^{\frac{3}{2}} = 19{,}091 \text{ Cu. ft. sec.}$$

Length of new dam = 677.37 Ft.

$$\text{Head} = \left(\frac{19{,}091}{3.33 \times 677.37}\right)^{\frac{2}{3}} = 4.15 \text{ ft.}$$

Elev. crest of new dam . . .	**305.00**
Height of crest for 4.15′ head	**—.50**
Elev. of theoretical crest . . .	**304.50**
Head	**4.15**
Elev. high water over new dam	**308.65**

Discharge through the 3 sluice gates, when water is at el. 308.65 = 3500 Sec. ft.

Fig. 119. Detail of Flashboard Arrangement of the Schaghticoke Dam.

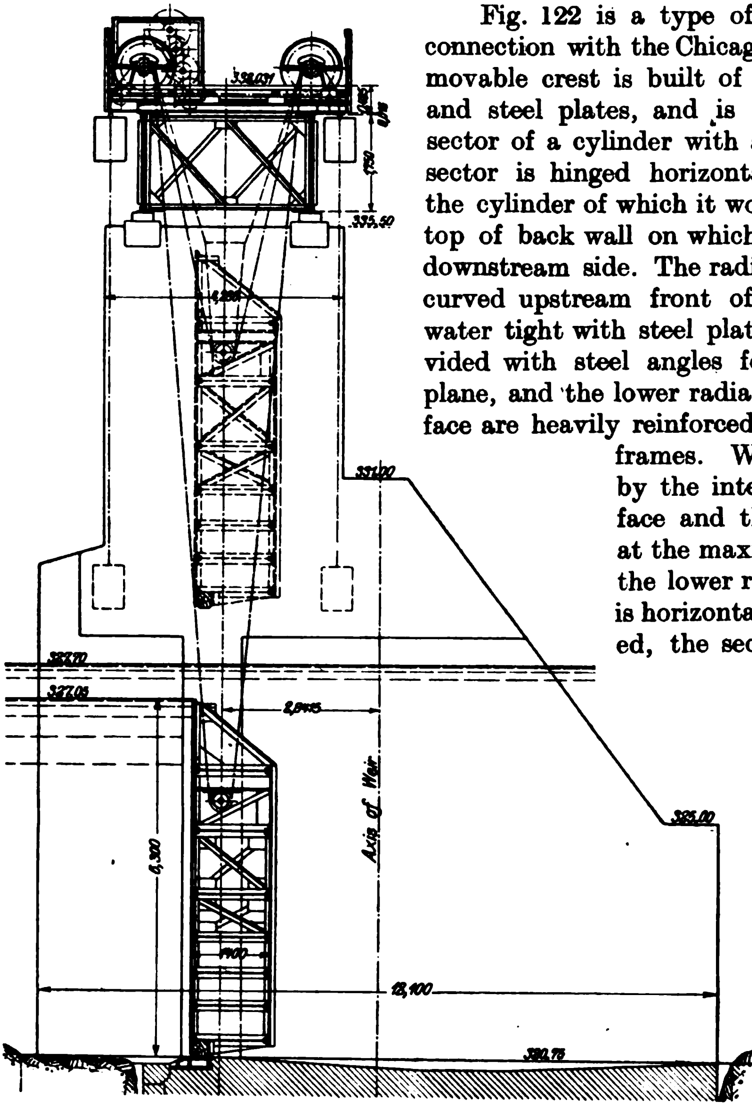


Fig. 120. Stoney Roller Sluice Gate.

Fig. 122 is a type of movable dam used in connection with the Chicago Drainage Canal. The movable crest is built of structural steel shapes and steel plates, and is practically a 45 degree sector of a cylinder with a 26 foot radius. Each sector is hinged horizontally along the axis of the cylinder of which it would form a part, to the top of back wall on which it is mounted on the downstream side. The radial deck plane and the curved upstream front of the sector are made water tight with steel plates, the deck being provided with steel angles for ice-skids. The deck plane, and the lower radial plane and the curved face are heavily reinforced by intermediate steel frames. When the crest formed by the intersection of the curved face and the radial deck plane is at the maximum operating height, the lower radial plane of the sector is horizontal. As the crest is lowered, the sector rotates on its axis and moves into a space in the concrete base, which is also approximately a sector of a cylinder, of about the same radius as that of the crest, the radial deck being horizontal when the crest is at its lowest position.

An automatic type of dam, which has been successfully used in Europe, and illustrated in fig. 123, is described as follows by E. Lauchli in the Engineering News:

"The automatic dam is composed of a movable part or panel resting at the bottom on a knife edge, and fastened at the top to a compensating roller made of steel plate and filled with concrete. This roller moves along a track located at each of its ends, and is so designed as to take, at any height of water upstream, a position such as will give the apron the inclination necessary for discharging a known amount of water, and in so doing will keep the upper pool at a constant fixed elevation.

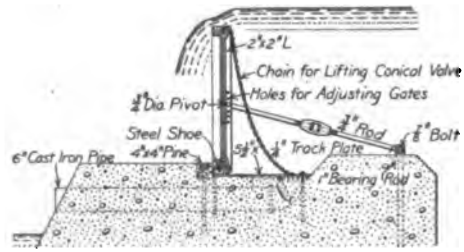


Fig. 121. Cross Section of Shutter Gate and Dam, Rio Guaso Irrigation System.

With the roller at its highest position the panel lies horizontally, and the full section is then available for discharging water. Any debris such as trees, or ice cakes etc., will pass over the dam without any difficulty, even during excessive floods, as the compensating roller is located high above extreme flood level.

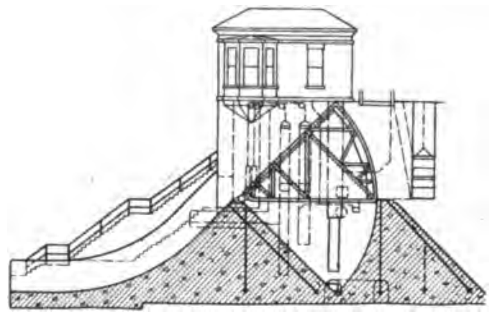
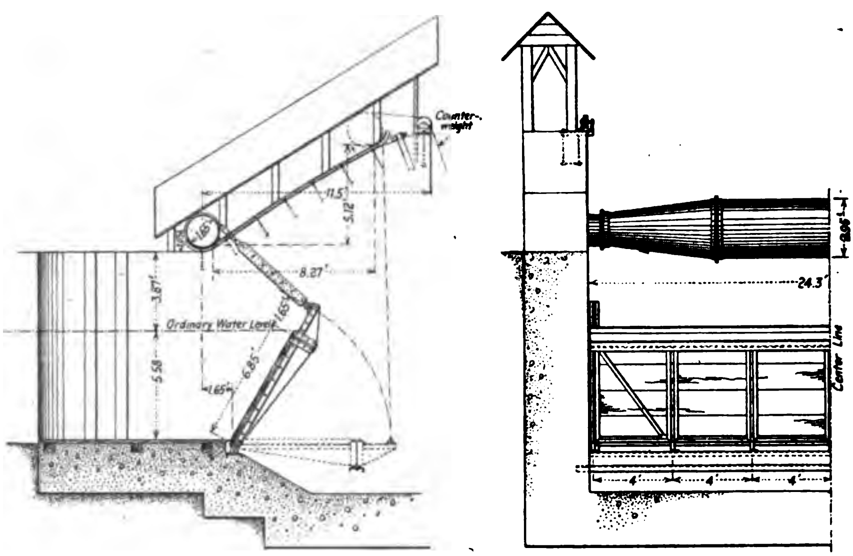


Fig. 122. Butterfly Dam, Chicago. Drainage Canal Power Plant.

The dam is located on the river Grafenauer Ohe, in Bavaria, and regulates the water level at the intake of a paper mill located at some distance from the power house. The dam has a panel 24.27 feet long, 6.85 ft. high and during normal water level will discharge 1400 cu. ft. per sec., while at flood time it will pass 3530 cu. ft. per sec. As shown in fig. 122, the main body of the dam is made of a wooden plank construction laid on a steel frame. The panel is connected with the compensating roller at each end by a flexible steel cable, wound around the roller end, and then fastened



Vertical. Section Half Front Elevation

Fig. 123. Details of small Automatic Dam.

at the upper end of the roller track to an eye bolt. A simple form of roof construction protects the roller track from rain and snow. The panel is made water tight at each extremity by means of galvanized sheet iron held tight against the abutments by water pressure. This type of construction has so far proved to be very effective as to water tightness.

It may be needless to point out that this type of dam can also be fitted to the crest of overflow dams of ordinary cross section and then fulfil the duty of movable flash boards."

88. Intakes.

Intakes may be of two classes depending on the system of carrying the water to the power plant. This may be done by means of pressure pipes, or by an open canal. In intakes of the first class the end of the pressure pipe is generally imbedded into the concrete; the masonry being calculated to resist against the overturning effect of the head water. Steel gates, wooden stop logs or butterfly valves are provided, together with a system of racks and a vent pipe to prevent the collapse of the pipe due to a possible vacuum when emptying the pipe line for inspection or repairs.

Figs. 124, 125 and 126 show the general layouts of pressure pipe intakes. If the intake discharges water into a canal, the upward water pressure under and along the opening must be considered, a thing that is not necessary in the case of pressure pipes, as the metal takes care of the upward thrust.

Fig. 127 shows the analysis of a canal intake of given dimensions, supposing the gate closed (no upward thrust). Still tail side water. Conditions are as follows:

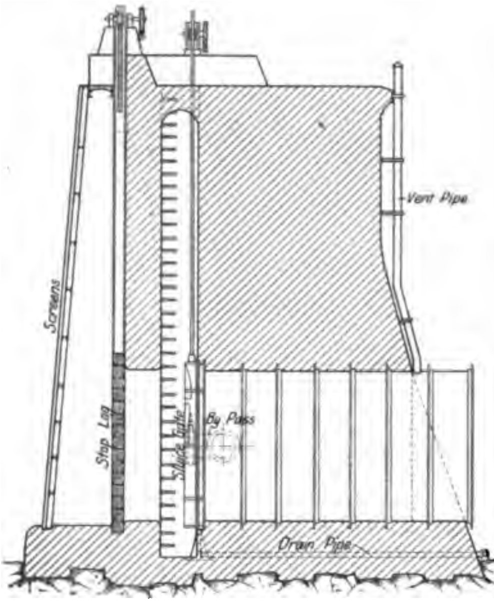


Fig. 124. Intake with Sluice Gate.

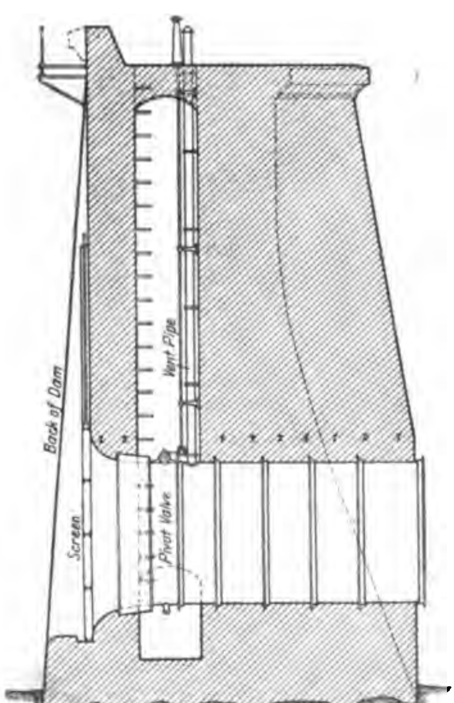


Fig. 125. Intake with Pivot Gate.

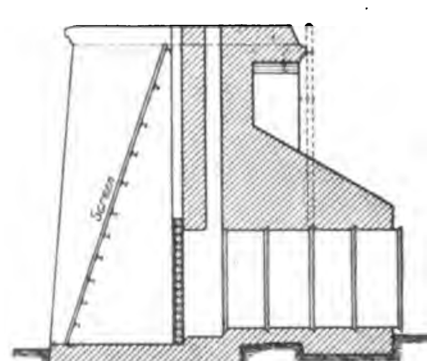


Fig. 126. Intake with Stop Log only.

Depth of water $= 41.5$ ft.

Water pressure per lineal foot at bottom $= 62.5 \times 41.5 = 2590$ lbs.

Arm of moment (water) $= \frac{41.5}{3} = 13.83$ ft.

Total pressure $= \frac{2590}{2} \times 41.5 = 53{,}700$ lbs.

Water pressure at bottom of downstream side $= 4.5 \times 62.5 = 281$ lbs.

Total water pressure tail side $= \frac{281}{2} \times 4.5 = 492$ say 500 lbs.

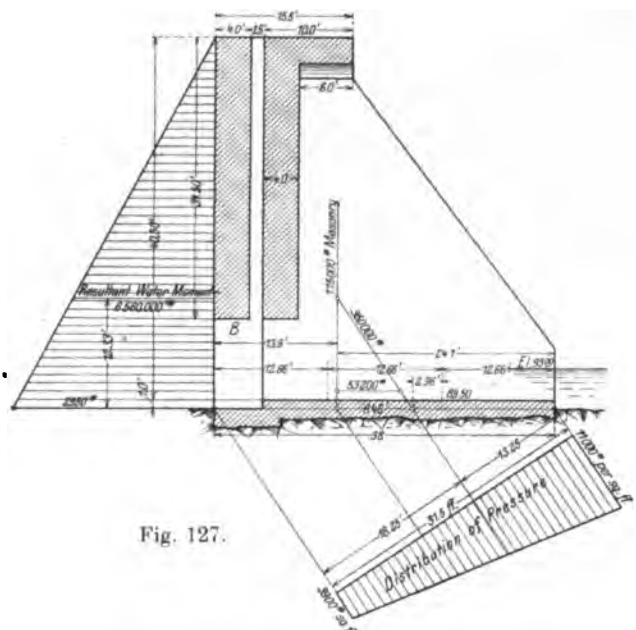


Fig. 127.

Resultant water pressure $= 53{,}700 - 500 = 53{,}200$ lbs.

For one bay, 10 ft. in width $= 53{,}200 \times 10 = 532{,}000$ lbs.

Arm of moment $= \frac{41.5 - 4.5}{3} = \frac{37}{3} = 12.33$ ft.

Resulting water moment $= 532{,}000 \times 12.33 = 6{,}560{,}000$ ft. lbs.

Width of base of intake $= 38$ ft.

Thickness of piers $= 4$ ft.

The factor of safety against overturning is

$$\frac{775{,}000 \times 24.1}{6{,}560{,}000} = 2.85.$$

The intake being located on uneven rock, possibilities of sliding are taken care of.

89. Fishways.

In many cases fish ladders must be built leading from the down stream side to the up stream side of the dam, and large enough to easily accomodate the migratory fish which will ascend thereby. The laws of many States require a fish ladder to be built.

Fig. 128 shows in detail a form of fishway approved by the United States Commission of Fish and Fisheries and is adaptable for construction through any sort of dam.

The directions of the Commission are as follow:

The slope of fishway should not be steeper than on a ratio of 1 vertical to 4 horizontal. Intake, or up-stream end of fishway should be amply large and placed not less than 1 foot lower than the crest of the dam.

Outlet should be below low-water level, and so located or constructed that fish are naturally led to it when ascending the stream.

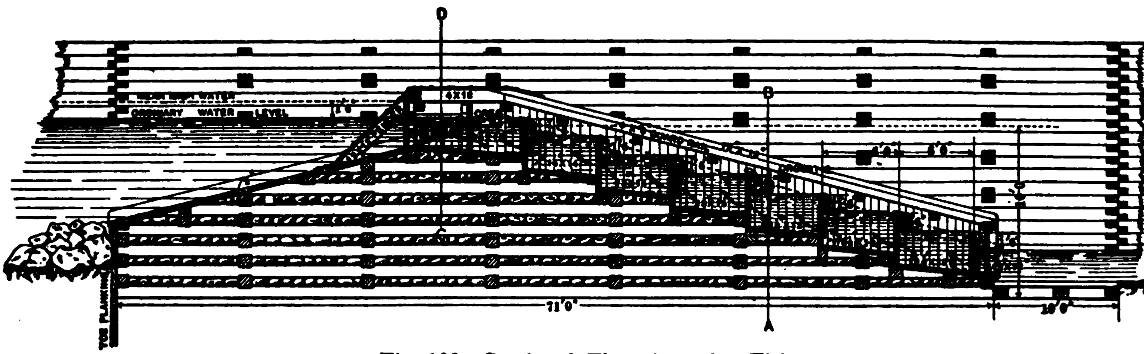


Fig. 128. Sectional Elevation of a Fishway.

There should be relatively deep water, with an unobstructed flow below the outlet of the fishway. An ample discharge of water should attract the fish to the outlet.

There should be plenty of light admitted in the fishway and its construction should be such as to be readily inspected and cleaned of any debris lodging therein.

The floor of the compartments should be laid slightly inclined, and the bulkheads somewhat obliquely across the fishway, so that the current of water passing through the compartments can more readily clear the same of sand, mud and rubbish.

There should be no regulating gates or other devices at the intake which necessitate the services of an attendant.

The apertures in the bulkheads should increase progressively from the lower to the upper ones, to insure overflow from compartment to compartment.

The flow of water should be abundant, forming small waterfalls over the bulkheads, so that the fish may either jump from one compartment to the next above or may dart through the apertures in the bulkheads.

While the flow of water through the apertures may reach a velocity of 10 feet per second, there will be relatively quiet water in the compartments, thus furnishing a resting place for the ascending fish.

To maintain the operation of the fishway at an average high water, the same as at the ordinary stage of the stream or river, the uppermost compartment is made somewhat longer, and a central bulkhead is inserted having its crest at high water level.

The fishway may be constructed of wood or masonry and iron; it may follow a straight line or be built in angles and curves as the local conditions may require.

The size of the fishway depends principally on the volume of water available, and can be made larger or smaller than shown on the plan. The hydraulic head between two successive compartments must be chosen so as to obtain a current velocity through the apertures of not to exceed 10 feet per second. At low stage of the stream or river, with the fishway flowing full, there should be a liberal discharge over the crest of the dam.

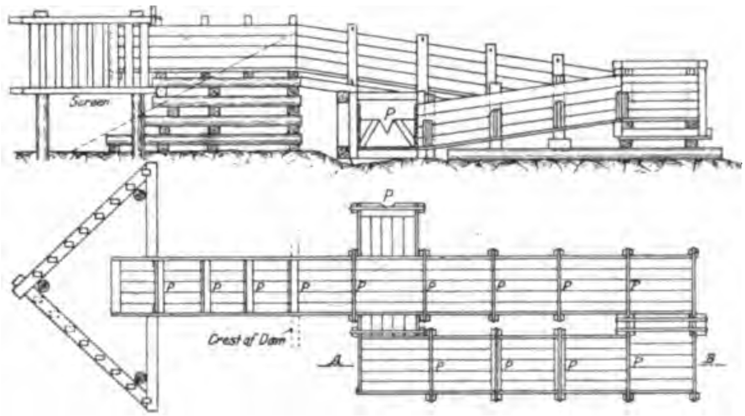


Fig. 129. Fishway of Fish Commission, State of Wisconsin.

The fishway should be built very strong and be well protected against the destructive effects of freshets, drift, ice etc.

A fishway of the Fish Commission, State of Wisconsin, is illustrated in fig. 129.

90. Amplitude of Backwater caused by a Dam.

If a dam is built across a stream, the depth of water just above the dam is increased. The longitudinal profile of the water surface is also influenced and takes the shape of a parabolic curve.

The following formula determines the amplitude of backwater for canals the depth of which is small as compared with the width, as is the case with many rivers of wide bed.

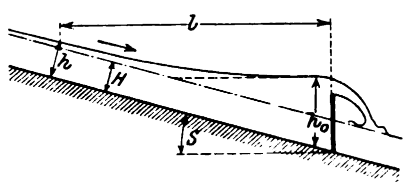


Fig. 130.

Let (fig. 130):

H = original depth,
h_0 = depth at the dam or weir,
h = depth at a distance l upstream,
l = distance upstream of the point h,
S = natural slope of water surface assumed parallel to stream bed,
C = coefficient in formula $v = C\sqrt{RS}$.

The distance l is given by the expression

$$l = \frac{h_0 - h}{S} + H\left(\frac{1}{S} - \frac{C^2}{g}\right)\left[\varphi\left(\frac{h}{H}\right) - \varphi\left(\frac{h_0}{H}\right)\right].$$

In practice, two cases may be met with in the investigations of backwater problems:

h and h_0 may be given, required to find l, or
l is given and one of the depths, required to find the other.

To solve such problems Table XL must be used to find values of the backwater function $\psi\left(\frac{h}{H}\right)$ for specific cases.

If $H = 4$; $h_0 = 10$; $h = 6$; $C = 80$; $S = 0.0001$; find l.

First find $\frac{H}{h_0} = 0.4$ for which the table gives $\psi\left(\frac{h_0}{H}\right) = 0.0821$ and $\frac{H}{h} = 0.6$ corresponding to $\psi\left(\frac{h}{H}\right) = 0.1980$.

Inserting these values in above formula gives

$$l = (10 - 6)\,10{,}000 + 4\left[10{,}000 - \frac{80^2}{32.16}\right](0.1980 - 0.0821) = 44{,}543 \text{ ft.}$$

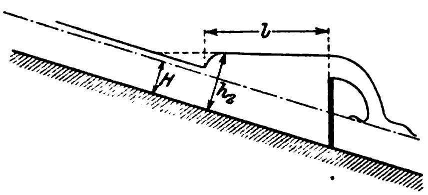


Fig. 131.

The inverse problem: to calculate h_0 or h when l and h_0 or h is given, can be solved only by successive trials. In case $S \gtreqless \frac{g}{C^2}$, a standing wave or hydraulic jump is formed, the height of which is determined by the expression

$$h_2 = 2\sqrt{h\frac{v^2}{2g}}$$

in which

$v =$ the mean velocity corresponding to h.

Should it be required to find the distance upstream to the point where the jump occurs, find this latter expression for h_2, substitute this value in the former, and solve for l (fig. 131).

In practice, when a case is met with in which the slope is small, it is better to use the following method.

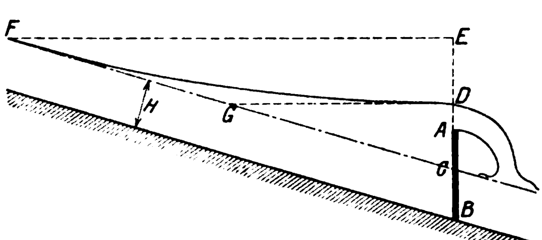


Fig. 132.

Let (fig. 132):

$A\,B =$ the dam or obstruction,
$F\,C =$ the slope of uniform flow.

The elevation of the point D, with respect to point A is first determined by Bazin's formula for discharge. All observations made on streams of small slope agree in the fact that the backwater ceases to be appreciable above a point situated on a horizontal line, through E when $ED = DC$.

Having determined the point F, the curve may be plotted by the equation of a parabola, the expression of which is

$$z = \frac{(lS - 2z_0)^2}{4z_0}$$

where

S = natural slope of water surface assumed parallel to stream-bed;
l = distance measured from dam to a point up-stream;
z_0 = the height CD just above the dam; and
z = the height of point up-stream which it is desired to locate.

By the use of this equation a series of values z may be determined giving the corresponding value of l. From such data the backwater curve may be plotted graphically.

Table XL.[1] *Values of the backwater function.*

$\frac{H}{h}$	$\psi\left(\frac{h}{H}\right)$	$\frac{H}{h}$	$\psi\left(\frac{h}{H}\right)$	$\frac{H}{h}$	$\psi\left(\frac{h}{H}\right)$	$\frac{H}{h}$	$\psi\left(\frac{h}{H}\right)$
1.	∞	0.948	0.8665	0.815	0.4454	0.52	0.1435
0.999	2.1834	.946	.8539	.810	.4367	.51	.1376
.998	1.9523	.944	.8418	.805	.4281	.50	.1318
.997	1.8172	.942	.8301	.800	.4198	.49	.1262
.996	1.7213	.940	.8188	.795	.4117	.48	.1207
.995	1.6469	.938	.8079	.790	.4039	.47	.1154
.994	1.5861	.936	.7973	.785	.3962	.46	.1102
.993	1.5348	.934	.7871	.780	.3886	.45	.1052
.992	1.4902	.932	.7772	.775	.3813	.44	.1003
.991	1.4510	.930	.7675	.770	.3741	.43	.0995
.990	1.4159	.928	.7581	.765	.3671	.42	.0909
.989	1.3841	.926	.7490	.760	.3603	.41	.0865
.988	1.3551	.924	.7401	.755	.3536	.40	.0821
.987	1.3284	.922	.7315	.750	.3470	.39	.0779
.986	1.3037	.920	.7231	.745	.3406	.38	.0738
.985	1.2807	.918	.7149	.740	.3343	.37	.0699
.984	1.2592	.916	.7069	.735	.3282	.36	.0660
.983	1.2390	.914	.6990	.730	.3221	.35	.0623
.982	1.2199	.912	.6914	.725	.3162	.34	.0587
.981	1.2019	.910	.6839	.720	.3104	.33	.0553
.980	1.1848	.908	.6766	.715	.3047	.32	.0519
.979	1.1686	.906	.6695	.710	.2991	.31	.0486
.978	1.1531	.904	.6625	.705	.2937	.30	.0455
.977	1.1383	.902	.6556	.70	.2883	.29	.0425
.976	1.1241	.900	.6489	.69	.2778	.28	.0395
.975	1.1105	.895	.6327	.68	.2677	.27	.0367
.974	1.0974	.890	.6173	.67	.2580	.26	.0340
.973	1.0848	.885	.6025	.66	.2486	.25	.0314
.972	1.0727	.880	.5884	.65	.2395	.24	.0290
.971	1.0610	.875	.5749	.64	.2306	.23	.0266
.970	1.0497	.870	.5619	.63	.2221	.22	.0243
.968	1.0282	.865	.5494	.62	.2138	.21	.0221
.966	1.0080	.860	.5374	61	.2058	.20	.0201
.964	0.9890	.855	.5258	60	.1980	.18	.0162
.962	.9709	.850	.5146	.59	.1905	.16	.0128
.960	.9539	.845	.5037	.58	.1832	.14	.0098
.958	.9376	.840	.4932	.57	.1761	.12	.0072
.956	.9221	.835	.4831	.56	.1692	.10	.0050
.954	.9073	.830	.4733	.55	.1625	.06	.0018
.952	.8931	.825	.4637	.54	.1560	.01	.0001
.950	.8795	.820	.4544	.53	.1497	.00	.0000

[1] From Bresse's "La Mecanique appliquée".

This approximate method may be applied to canals and rivers, and notwithstanding accidental irregularities due to the influence of slope, river cross section and river bed, the curve as plotted will be found sufficiently accurate for practical purposes. A case of standing wave of interesting character is the one that may be formed between bridge piers. If a steel highway or railway bridge crosses a stream just above the dam, it is of importance to calculate the actual clearance between the bottom chord of the bridge and the high water line, in case the dam is to be built in close proximity to such a structure. Floating ice or other debris may cause damage by coming into contact with it, a possible occurrence due to the combined conditions of backwater caused by the dam and standing wave owing to contraction of river cross section.

The height of this standing wave as given by M. Bresse is

$$h = 1.1 \frac{Q^2}{2g}\left[\frac{1}{w^2 l^2 H^2} - \frac{1}{L^2 (H+h)^2}\right]$$

in which

l = width of contracted channel or clearance between piers;
L = distance center to center of piers;
Q = flow with reference to width L;
h = height of wave;
H = depth of water under bridge;
w = a coefficient of contraction, taken as 0.80, for square piers, and 0.90 for piers sharped off in a triangular nose on the upstream side.

Chapter VI. Turbines.

91. Classification of Turbines.

Turbines may be classified in several different ways. Generally speaking, there are three types of turbines:

Reaction or pressure turbines;
Impulse or tangential turbines; and
Limit turbines.

The turbine of the first type is completely filled with water, or drowned, and the water acts by its own pressure (Fourneyron, Jonval Francis); turbines of the second type are put in action by the head which is converted in velocity (Girard, Pelton, etc.); limit turbines form the dividing line between reaction and impulse wheels and combining many advantages of both types (Haenel) they may be considered as impulse turbines without free deviation. Another classification of turbines may be made according to the direction of the flow of water:

Outward radial flow turbines (Fourneyron, Cadiat)
Inward flow (Francis, Scheile)
Parallel or axial flow (Jonval, Girard), direction of the flow being parallel to turbine axis; and
Mixed flow or American turbine (Swain), admission being radial inward and discharge axial downward (Fig. 133).

The Fourneyron turbine, diagramatically represented in fig. 134. is one in which the concentric distributor is placed inside the runner. The guides *G* of the distributor are attached to a plate. Such turbines may discharge into the atmosphere, or may also be drowned, although the use of a draft tube is not convenient, nevertheless favorable results are secured with spiral draft chests. The Fourneyron type has found application on a large scale at the plant of the Niagara Falls Water Power Co. (Fig. 135). They are of a capacity of 5000 HP at an efficiency of 75 $^0/_0$, discharge 430 cu. ft. per sec. and have a speed of 250 r. p. m. The buckets are divided vertically into three sections in order to increase the part gate efficiency.

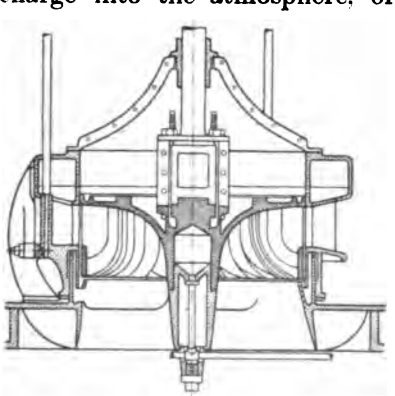


Fig. 133. Section of Swain Turbine.

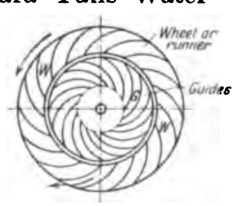


Fig. 134. Fourneyron Turbine.
Radial Outward Flow.

If the distributor is placed outside and is concentric with the runner, the flow is inward, and the best known wheel of this class is the Francis turbine, diagramatically illustrated in fig. 136. Fig. 137 shows a vertical shaft turbine built by the firm Voith for the Kykkelsrud plant, Norway. It operates under a head of 52.5 to 62.5 feet, with a water consumption of 670 to 530 cu. ft. per sec. and a speed of 150 r. p. m. The capacity of each wheel is 3000 HP, the water supply into the turbine being controlled by clam shell gates. The turbine built by Kolben & Co., Prague, Bohemia, and installed at the Wiesberg Plant, Tyrol, is a compound turbine of the Francis spiral type (fig. 138). It consists of two turbines mounted on the same shaft, so connected that the discharge of one becomes the supply of the other thus reducing the head on each turbine from 285 to 142.5 feet. The spiral casing of each turbine is of cast iron and made in two parts with an inlet of 33.5 inches diameter. Both runners are 41.25 inches in diameter and of cast steel, and have 19 vanes. With a water consumption of 88 cu. ft. per sec., the compound turbine developed 2260 HP, making 342 r. p. m., and giving with a gate opening of 30, 60 and 90 per cent, efficiencies of 67, 81 and 86 per cent respectively. Large reaction turbines were installed in connection with the new development on the White River, California, their rating being 20400 HP under a head of 480 feet, and a speed of 360 r. p. m. The turbine runner is of the high pressure Francis type with horizontal shaft. Water is admitted to the runner through a cast steel spiral casing. The runner which is also of cast steel, divides the water into two lines of flow, and it is discharged from the wheel by two quarter turns to separate draft tubes. The flow of water to the turbine is controlled by a cast steel butterfly valve over 7 feet in diameter. The shaft carrying the turbine runner is nearly 2 feet in diameter and the bearings for it about 16 inches in diameter. In the *"axial" or Jonval turbine* (fig. 139) the distributor and runner are placed one above the other about the same axis. The discharge may take place either into the air, into the tail water, or into a draft tube. Fig. 140 shows the *Geylin-Jonval* turbine as manufactured by the R. D. Wood Co. of Philadelphia. *W* is the runner, *B* represents the buckets, *g* are the guides conveying the water to the buckets. The wheel here illustrated has double inlets that are closed by the double cylinder gates *C G*. This gate closes up against the hood *C* by means of the rods *r*, which connect with the governor mechanism. A tangential wheel of the Girard type is shown in fig. 141. The runner is represented by *W*, *BB* are the buckets, and *G* the gate by which all or a portion of the guide passages may be closed. The gate *G* is connected by the gearings *G r* with the rod *r*, which is connected through the rocker arm with the governor mechanism. Fig. 142, shows a plan and

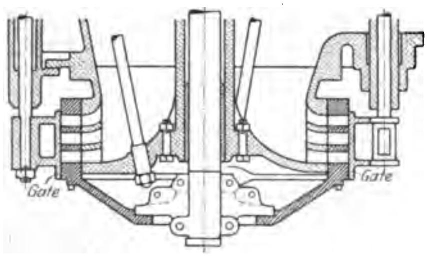


Fig. 135. Lower Part of 5000-Horse-Power-Turbine, Power-House No. 1, Niagara Falls Power Company.

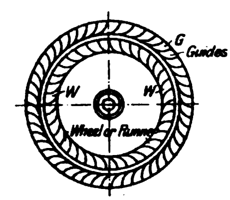


Fig. 136. Francis Turbine Radial Inward Flow.

section of an improved American turbine manufactured by the Dayton Globe Iron Works Co. *W* is the crown and hub of the wheel; *B* are the buckets; *G* are the wicket gates which control the admission of the water; these are operated by the ring *G r* connected through the shaft *P* with the governor. Mr. Gelpke classifies the Pelton wheels as follows:

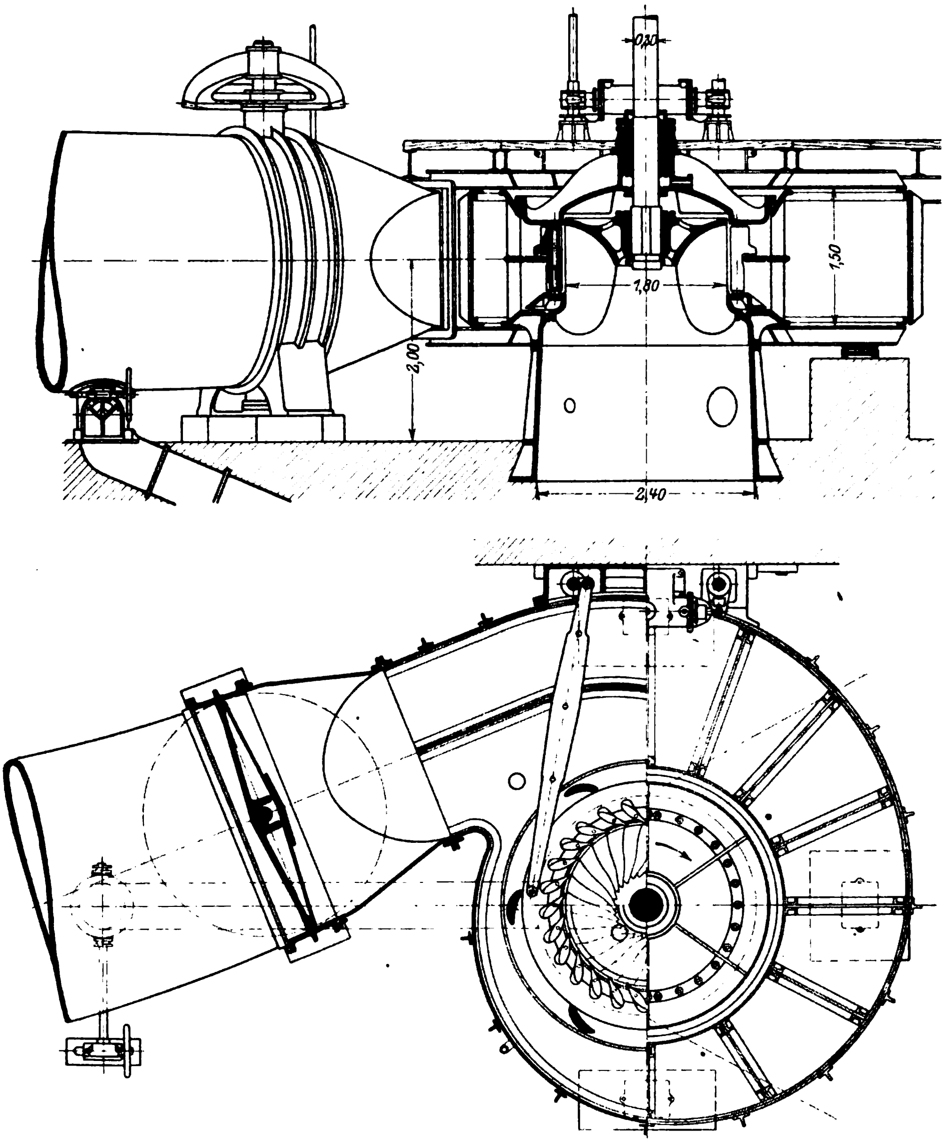


Fig. 137. Vertical Shaft Francis Turbine.

1. With circular nozzles:

a) Nozzle closed by means of a sliding cut-off.
b) Nozzle closed by means of a needle placed within the nozzle and moving in the direction of its axis (needle valve).

c) Nozzle closed by means of an outer sheath surrounding a stationary needle and moving in relation to it.

2. *With nozzles square or rectangular in cross section:*

a) Nozzle closed by means of a sliding cut-off, just as in the case of circular nozzles.

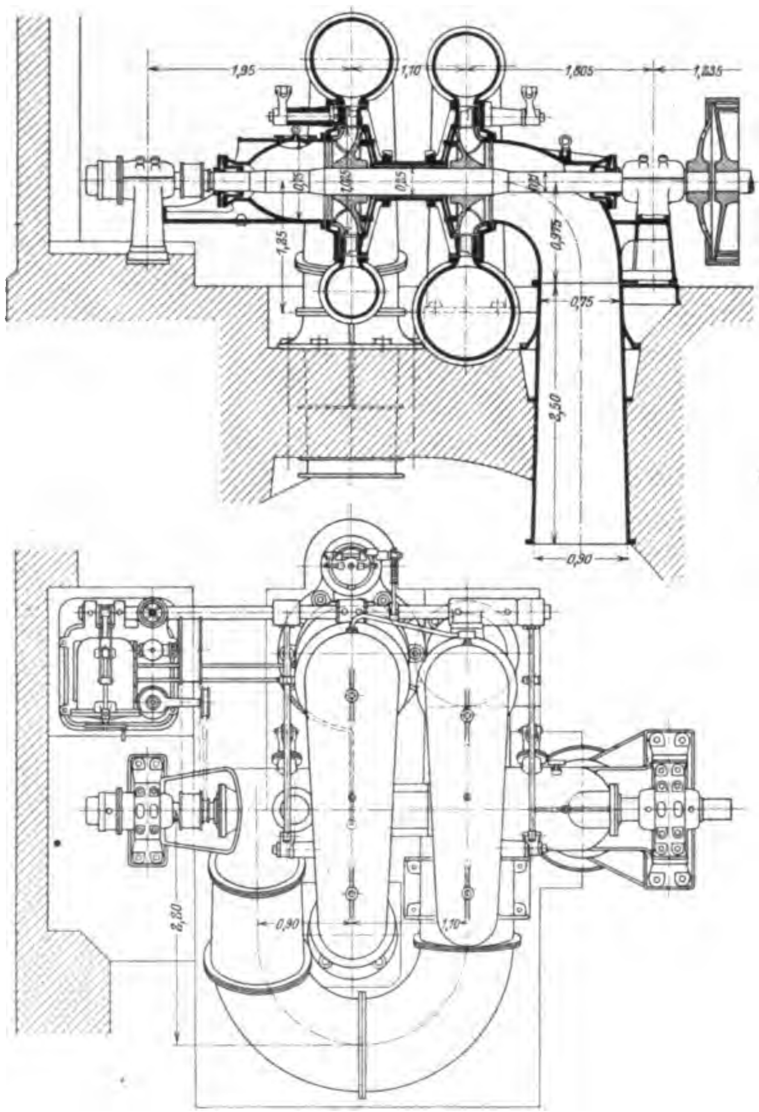


Fig. 138. Compound Francis Turbine.

b) Nozzle closed by a contraction of the thickness of the jet, this being usually accomplished by revolving the upper side of the nozzle around a pivot. or a parallel motion in guides.

c) Nozzle closed by a contraction of the width of the jet, the two sides of the nozzle being moved toward each other either by swinging them or by their parallel motion, the upper and lower sides of the nozzle remaining stationary.

Fig. 143 shows a Bell spoon turbine and governor of the Obermatt plant, Switzerland.

The type of turbine to be employed in each case is to be as follows according to Thurso:

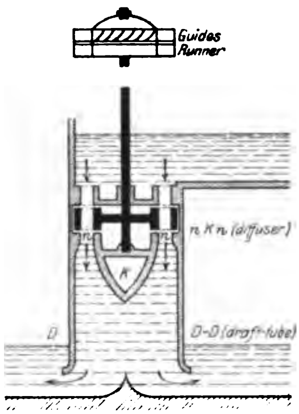


Fig. 139. Jonval Turbine Axial Flow.

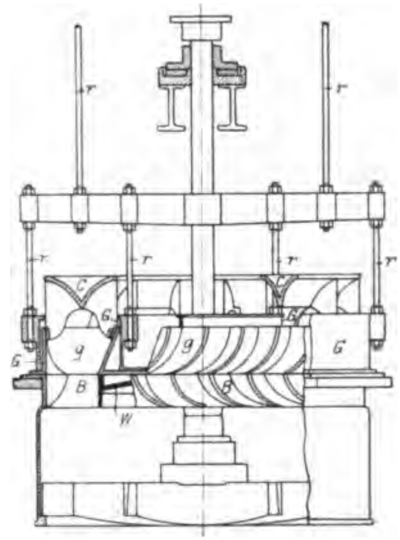


Fig. 140. Vertical Shaft Geylin-Jonval Turbine.

1. Low Heads, say up to 40 feet: American type of turbine (i. e. of the "inward and downward" variety). with horizontal or vertical shaft in open flume or case, nearly always with draft tube. For heads up to about this limit, the American type of turbine has the great advantage over all other turbine types in common use. that it gives the greatest number of revolutions for a given head and power developed, or the greatest power for a given head and diameter of runner.

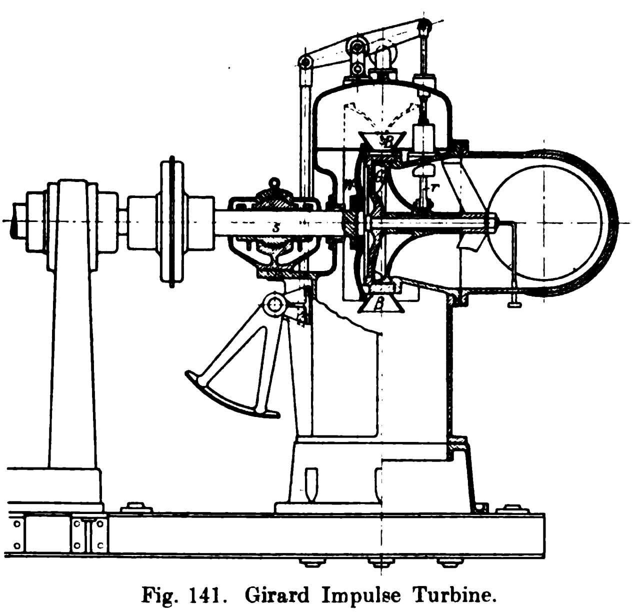


Fig. 141. Girard Impulse Turbine.

2. Medium Heads, say from 40 to 300 or 400 feet: Radial inward flow reaction or Francis turbine, with horizontal shaft, and concentric or spiral cast iron case with draft tube.

3. High Heads, say above 300 or 400 feet: Impulse wheel of the usual type (Pelton); or radial outward flow, segmental-feed, free deviation (a Girard impulse wheel); or a combination of both, with horizontal shaft and cast or wrought-iron case, often with draft tube.

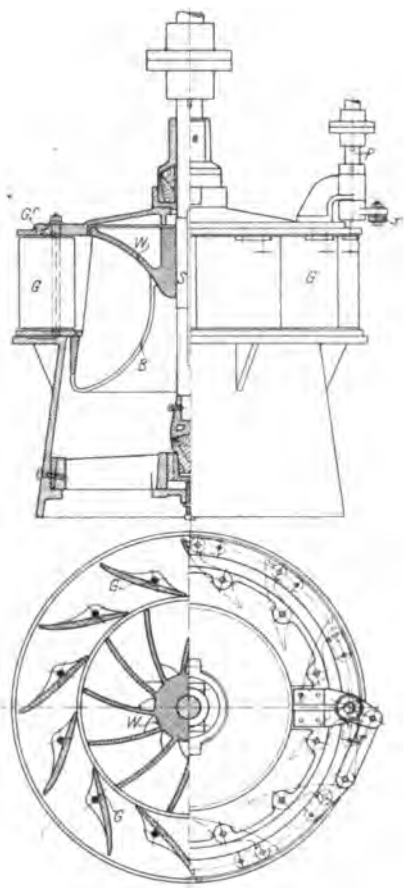


Fig. 142. Section and Plan of Improved New American Turbine.

92. Relative Advantages of Reaction and Impulse Wheels.

In cases of low or moderate heads, and when there is an ample supply of water, the reaction turbine is better adapted. Reaction turbines are generally drowned, that is to say, set below the tail water level, or installed in connection with a draft tube, and in this case the effective head extends from head water level to tail water level, a condition which cannot be fulfilled with impulse wheels, which discharge into the air and must be set far enough above tail water to be out of reach of back water which varies with the amount of flow, and if the amplitude of variation is important, a considerable percentage of head is lost.

On the other hand, the diameter of an impulse wheel may be increased at convenience, in order to obtain a moderate speed, which is a distinct advantage, especially considering that alternators are generally direct connected to the turbine shaft; they are therefore preferred for high heads. In the same case, the speed of a reaction turbine would result inconveniently great, although it may be reduced by lessening the gate opening, a course which inevitably results in material loss of energy. When a wheel must operate under a variable load it also becomes necessary to maintain a constant speed, and it is imperative to reduce the discharge of the wheel. This also involves a considerable loss of energy. Experience shows that a turbine of the impulse type has a marked advantage as far as efficiency and low supply is concerned. To prevent the loss of head incurred by the part gate operation, the turbine runner may be divided into several portions horizontally, one or more of which may be entirely closed off by the gate and the remaining partitions continuing in operation. The efficiency would remain about the same as at full gate opening. Fig. 144 shows a single three story inflow reaction turbine having

a runner of conical shape with the small diameter at the top. The bottom and middle story of the runner are used at all stages of the water, while the top one is used in addition during flood times. when the head is at a minimum.

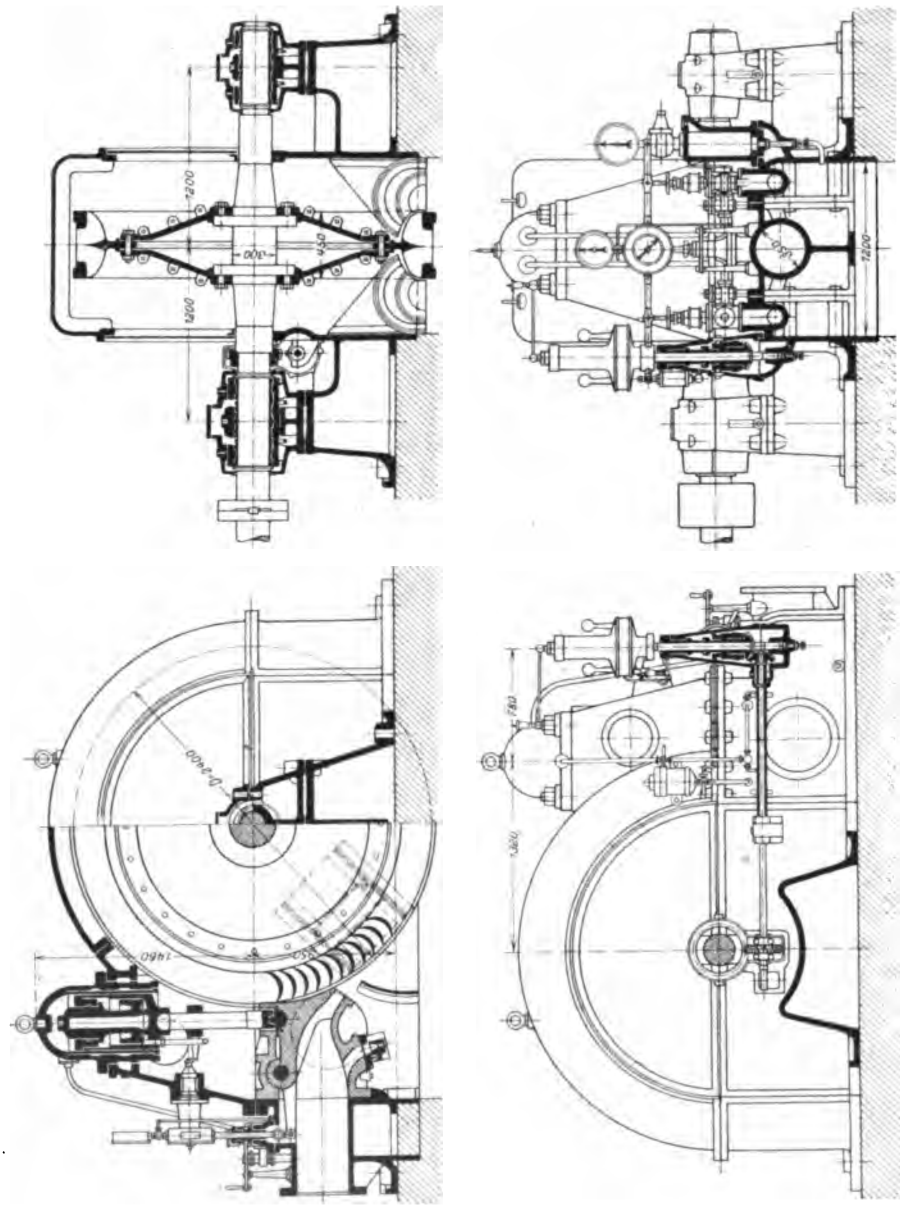


Fig. 143. Bell Spoon Wheel Turbine and Governor, Obermatt Plant, Switzerland.

93. Horizontal and Vertical Shaft Turbines.

These two types have relative advantages and disadvantages. the most important ones being herein enumerated. Horizontal shaft turbines, as well as vertical shaft turbines, are used for low and medium heads. The ma-

chines of the first type are generally placed in the generator room, offering thereby every facility for installation inspection and repairs; they are very well suited for direct coupling, and as a matter of fact, are cheaper to install than the equivalent vertical shaft type. However the whole weight of the machine, as well as the force of the horizontal component of the flowing water, are transmitted to the bearings. with the result that it is necessary to design and proportion the foundations with care. Further the entrance of the water to the turbine case is effected by the pressure

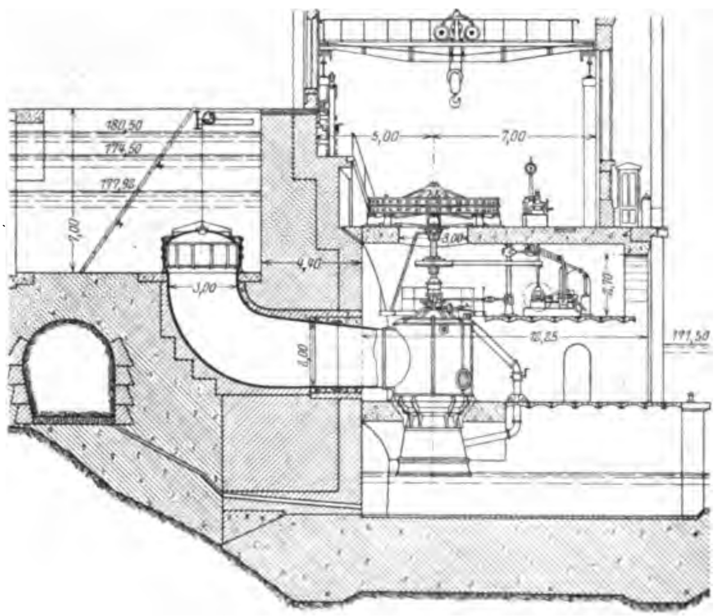


Fig. 144. Single Three-Story Inflow Turbine, Lyons Plant-France.

pipe extending into the generator room: the electric machinery is exposed to wetting, and the room may be flooded in event of the breaking of the pressure pipe, due to water hammer effect. This extreme case, however, seldom occurs in practice. The best security is obtained by placing the extending part of the pressure pipe below the generator floor. The vertical shaft turbine has the whole weight of the machine supported by a pivot. In cases of direct connection to the generator, the weight of the revolving part is added. This thrust is best counter-balanced by either the water pressure itself, or by means of oil, also under pressure. Vertical shaft wheels are in most instances submerged, therefore, difficulty results in installation, inspection and repairs. This kind of wheel has become nevertheless of general use; its adoption reduces the size of the power house floor area, a factor that greatly influences first cost, even if it demands more excavation.

94. Turbine Calculations. General Notation.

= Volume of water in cubic feet per second,

H = effective head in feet,

s = speed in revolutions per minute.

$\omega = \frac{\pi s}{30} = \frac{v}{r}$ = angular velocity in feet per second,

ε = efficiency in percentage,

W = weight of one cubic feet of water,

P_t = theoretical horse power,

$P_e = \varepsilon P_t$ = hydraulic HP delivered to the turbine shaft,

h = height of head water above center line of runner,

b = atmospheric pressure per square foot,

p = pressure head of water on guide orifices at the gate openings, as would be indicated by piezometer tubes if inserted at such points,

h' = height of tail water below center plane of runner; or suction head,

$h + h' = H$ = effective head,

V = absolute velocity of water discharge from distributor,

v = velocity in feet, of the inside circumference of runner periphery,

w = relative velocity in feet with which the water enters the runner,

r = internal radius of the runner,

r_0 = external radius of runner,

v' = velocity in feet of the outside circumference of the runner periphery,

w' = relative velocity in feet with which the water leaves the runner,

α = terminal angle of the distributor bucket, or the angle formed by v and V,

β = terminal angle of the runner bucket with the external circumference of runner,

γ = angle made by velocities v and w.

95. Calculation of Outward Radial Flow Turbines. (Fig. 145.)

Bernouilli's theorem is given by the expression

$$V = \sqrt{2g\left(h + \frac{b - p}{W}\right)} \quad \ldots\ldots\ldots\ldots (1)$$

These fundamental equations are known also:

$$v = \omega r; \qquad v' = \omega r_0.$$

The energy developed by the water has for expression:

$$\frac{1}{2} m (v'^2 - v^2)$$

and the amount of increase of this energy becomes

$$\frac{1}{2} m (w'^2 - w^2).$$

Neglecting the effects of friction and gravity, and the water motion being horizontal, the following relation is found:

$$w'^2 = w^2 + v'^2 - v^2 + 2g\left[\frac{p-b}{W} - h'\right] \quad \ldots\ldots (2)$$

The trigonometric relations at the points A and B are respectively

$$w^2 = V^2 + v^2 - 2vV\cos\alpha \quad \ldots\ldots\ldots (3)$$

and

$$V'^2 = w'^2 + v'^2 - 2v'w'\cos\beta \quad \ldots\ldots\ldots (4)$$

Making $w' = v'$ and introducing these values in equations (1), (2), (3), we find

$$2gH - 2vV\cos\alpha = 0$$

from which

$$v = \frac{gH}{V\cos\alpha} \quad \ldots\ldots\ldots\ldots (5)$$

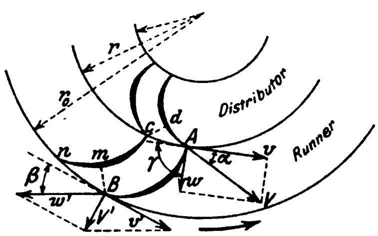


Fig. 145.

Calling Bm and cd the perpendiculars on the guide and runner buckets and considering the quantity of water at entrance and at discharge as being equal, the following relations are found

$$Ac\sin\alpha V = Bn\sin\beta v'$$

or

$$r^2\sin\alpha V = r_0^2\sin\beta v \quad \ldots (6)$$

Combining equations (5) and (6) gives:

$$V = \frac{r_0}{r}\sqrt{\frac{2gH\sin\beta}{\sin 2\alpha}} \quad \ldots\ldots\ldots\ldots (7)$$

$$v = \frac{r}{r_0}\sqrt{\frac{2hH\tan\alpha}{2\sin\beta}} \quad \ldots\ldots\ldots (8)$$

By means of (8), the angular velocity is determined.
Making $w' = v'$ in equation (4), gives:

$$V'^2 = 2v'^2(1 - \cos\beta),$$

$$= 2\frac{r_0^2}{r^2}v^2(1 - \cos\beta),$$

$$= 2gH\tan\alpha\frac{1-\cos\beta}{\sin\beta}$$

$$= 2gH\tan\alpha\tan\frac{\beta}{2} \quad \ldots\ldots\ldots\ldots (9)$$

From this equation the efficiency of the wheel may be determined. The work imparted to the wheel is theoretically

$$WH - \frac{WV'^2}{2g},$$

the theoretical horse power being WH, the efficiency will be:

$$\varepsilon = \frac{WH}{WH - \frac{WV'^2}{2g}} = 1 - \frac{V'^2}{2gH}$$

or

$$\varepsilon = 1 - \tan\alpha \tan\frac{\beta}{2}.$$

If we call d and d_1 the entrance and exit depths respectively, the expression for efficiency becomes

$$\varepsilon = 1 - \frac{d}{d_1}\tan\alpha\tan\frac{\beta}{2}.$$

The general condition that for maximum efficiency the water must enter tangentially to the vanes, will be fulfilled when

$$\frac{v}{V} = \frac{\sin(\gamma - \alpha)}{\sin\gamma}$$

equation which combined with (6) gives

$$\frac{r_0^2}{r^2} = \frac{\sin\alpha\sin\gamma}{\sin\beta\sin(\gamma-\alpha)}.$$

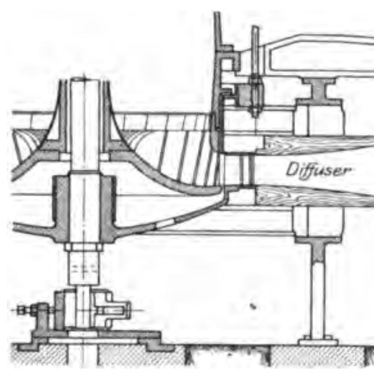


Fig. 146. Boyden Diffuser.

It will be remarked that when the smaller are the angles α and β, the greater will be the efficiency of the turbine. According to M. Mueller[1], the losses of energy in reaction turbines vary from 17 °/₀ to 27 °/₀ as follows:

Hydraulic resistance	10 to 14 °/₀
Velocity of water discharged from turbine . .	3 to 7 °/₀
Leakage at clearance	2 to 3 °/₀
Shaft friction	2 to 3 °/₀
Total losses:	17 to 27 °/₀

In order that the above conclusions be valid, it is necessary that the water discharging from the guide buckets meet the vanes of the runner buckets without shock or vacuum, and therefore the changes in direction must be gradual and formed by easy curves. Diffusers are often used in connection with outward radial flow turbines. The purpose of the diffuser (fig. 146) is to ensure a gradually enlarged opening, thereby gradually decreasing the velocity with which the water leaves the wheel.

[1] Mueller-Francis Turbinen-Hanover.

96. Calculation of Inward Radial Flow Turbines. (Fig. 147.)

The general condition of maximum efficiency is the same as for the outward flow turbines:

$$\frac{v}{V} = \frac{\sin(\gamma + \alpha)}{\sin \gamma}.$$

The trigonometric relations at A and B, are as before:

$$w^2 = V^2 + v^2 - 2\,V\,v\,\cos\,\alpha$$
$$V'^2 = w'^2 + v'^2 - 2\,w'\,v'\,\cos\,\beta.$$

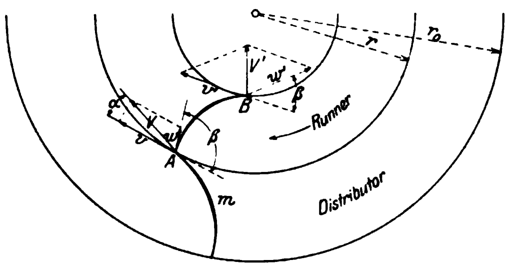


Fig. 147.

Again, angle β must be as small as possible, and making $w' = v'$ we have

$$v = \frac{g\,H}{V\cos\alpha}.$$

In order that there will be no reaction, we must have

$$\frac{p - b}{W} = h'.$$

When the turbine works without reaction, the following relations are found

$$V = \sqrt{2\,g\,H} \quad \text{and} \quad w = v$$

from which

$$\frac{V}{v} = \frac{\cos\,\alpha}{2}.$$

The ratio $\frac{V}{v}$ depends entirely upon the angle α, and allows the inward flow turbines to be classified according to speed; slow speed turbines satisfy the condition

$$\frac{v}{V} \leq 0.60$$

and high speed turbines

$$\frac{v}{V} \simeq 1.$$

If the turbine discharges into the air at a height h_a for instance, the effective head is reduced by that amount and the efficiency becomes

$$\varepsilon = \left[1 - \frac{V'^2}{2\,g\,H}\right] - \frac{h_a}{H}.$$

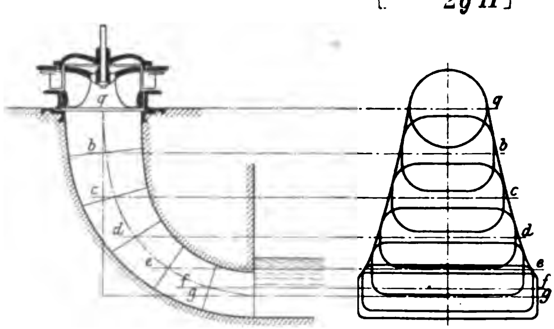


Fig. 148. Reaction Wheel with Concrete Draft Tube.

It is preferable, therefore, that the turbine be drowned or adapted to a draft tube. As in the outward flow turbines, it is essential that the residual velocity of the water in leaving the buckets be a minimum. The relation of a radial inward flow reaction turbine to a concrete draft tube is shown

in fig. 148[1]. The cross section of the draft tube gradually changes at successive points in order that the velocity of the flow may be gradually decreased from the point of discharge of the turbine to the tail water canal. The changes in absolute velocity of the water into and through the turbine, and finally through the draft tube are also shown in the diagram of fig. 149. The absolute velocity is a maximum at or near the point where the water enters the runner and is decreased as greatly as possible at the point of discharge into the draft tube.

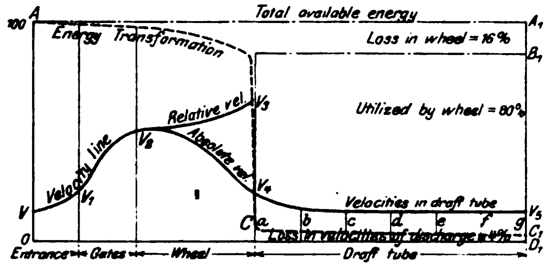


Fig. 149. Graphical Relation of Velocity and Energy in the Flow through a Reaction Turbine with Draft Tube.

Data for design: Call u the velotity of the water in a circular section of radius r_1. The condition to be satisfied is

$$u = 0.25\sqrt{2gH} \quad \text{to} \quad 0.15\sqrt{2gH}.$$

Assuming u, and knowing Q and H:

$$r_1 = \sqrt{\frac{Q}{\pi u}}.$$

Take

$$r = 1.5\, r_1 \quad \text{to} \quad 2\, r_1$$

and then

$$\frac{r_o}{r} \approx 1.15.$$

The total horizontal length of opening of the distributor will be:

$$2\pi r \sin\alpha \times m k.$$

For the runner:

$$2\pi r_1 \sin\beta \times m' k'$$

where k and k' take care of the thickness of distributor and runner vanes m and m' are contraction coefficients varying from 0.85 to 0.95. Calling p and p_1, the depth of opening of the orifices, we have:

$$Q = m k \times 2\pi r \sin\alpha \times p V \quad \ldots\ldots\ldots (1)$$
$$Q = m' k' \times 2\pi r_1 \sin\beta \times p_1 w'.$$

As $V = \sqrt{2gH}$, and r being known, the following expression is deduced from (1):

$$Q = m k \times 2\pi r \sin\alpha \times p\sqrt{2gH} \quad \ldots\ldots\ldots (2)$$

Let

z = number of vanes; and

t = thickness of vanes; then

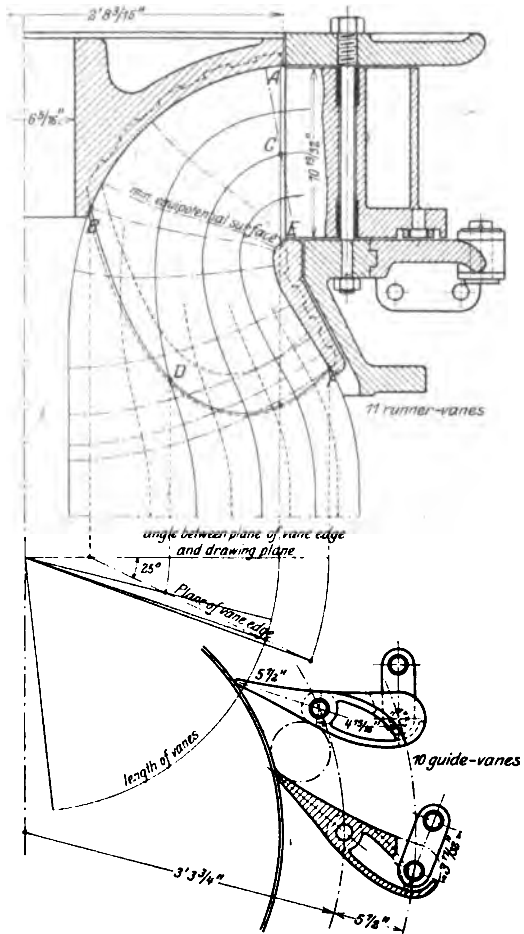
$$Q = m(2\pi r \sin\alpha - z t) p \sqrt{2gH}.$$

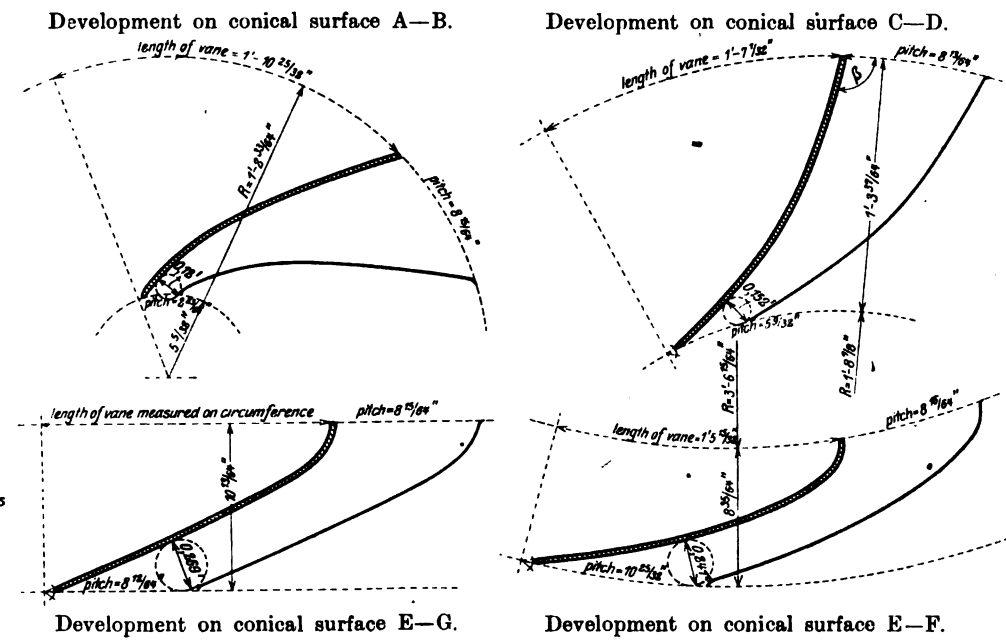
[1] Victor Gelpke. Turbinen und Turbinenanlagen.

Fig. 150. Form of Runner-Vanes for Inward-Flow Turbine.

Built for:

Head 24.5 ft., Volume of Water 87 cu ft., Revolutions 220.
Head 1 ft., Volume oft Water 17.5 cu ft., Revolutions 44.5.





The value of p in (2), depends on α. Now as

$$v = \frac{2\cos\alpha}{V} \quad \text{and} \quad s = \frac{60\,v}{2\,\pi\,r},$$

we have finally

$$s = \frac{60\,V}{4\,\pi\,r\cos\alpha}$$

and as

$$V = \sqrt{2gH}$$

s and v are easily determined. Fig. 150,[1] shows the form of runner vanes for an inward flow turbine built for a head of 24.5 feet, discharging 87 cu. ft. per sec., and having a speed of 220 r. p. m.

97. Mixed Flow or American Turbines. (Fig. 151.)

The fundamental equation is:

$$V^2 = 2g\,[H - h']$$

and for reaction turbines

$$w^2 = V^2 + v^2 - 2\,V\,v\cos\alpha$$
$$w'^2 = w^2 + v'^2 - v^2 + 2g\,h'$$
$$V'^2 = w'^2 + v'^2 - 2\,w'\,v'\cos\gamma.$$

If $w = v$, we have

$$\gamma + 2\,\alpha = 180^0$$
$$\gamma = 180^0 - 2\,\alpha.$$

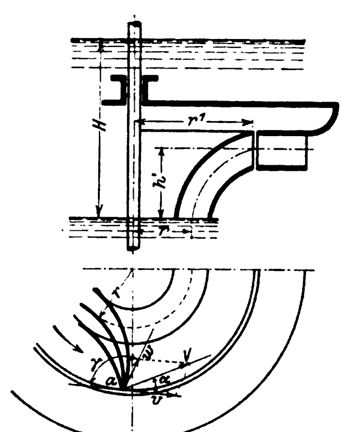


Fig. 151.

As α varies from 25^0 to 30^0, γ varies from 130^0 to 120^0 and the absolute velocity of the water at the exit of the distributor vanes is

$$V = \sqrt{2gH}.$$

If the turbine works under reaction, the following conditions are obtained:

$$\gamma < 120^0 \quad \text{and} \quad V < \sqrt{2gH}.$$

In most cases

$$\gamma = 120^0.$$

The degree of reaction has for value

$$R = \frac{V}{\sqrt{2gH}}$$

and the efficiency is

$$\varepsilon = 1 - \frac{V'^2}{2gH}.$$

This class of turbine should always be drowned, or installed in connection with a draft tube.

98. Taylor's[2] Simplified Formulae.

Although the Taylor formula has been replaced by the characteristic known as specific speed, it is in interesting to know that this formula is equivalent to specific speed and this indicates that the I. P. Morris Co. were

[1] Gelpke-Van Cleve. Hydraulic turbines.
H. B. Taylor. Hydraulic Engineer. The I. P. Morris Co. Phila.

familiar with the equivalent of specific speed before specific speed was introduced by German engineers.

Let H = net operating head, in feet

Q = discharge in second feet;

P_e = horse power of one runner;

ε = efficiency of wheel, written 0.75, 0.80, etc.;

D = diameter of tips of wheel vanes, measured on center line of distributor, in feet,

D_1 = diameter of discharge space of runner measured across draft tube or draft chest, in feet,

$f = \frac{D}{D_1}$, a characteristic constant for a given runner called flare;

u = the velocity of the water passing through the space of diameter equal to $f\,d$, found by dividing the quantity discharged by the area, thus giving the mean velocity.

$L = \frac{u^2}{2\,g}$, the velocity head corresponding to u, the "outflow loss from runner".

s = speed in r. p. m.

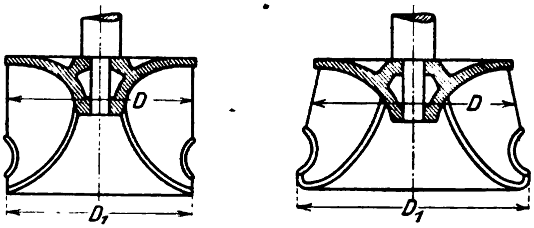


Fig. 152.

The actual kinetic energy in the water leaving the wheel is greater than $\frac{u^2}{2\,g}$, on account of the variation of the velocity across this space; this energy is not entirely lost to the wheel, however, since part of it may be regained by reducing the draft tube velocity at the discharge end.

For wheels of similar type and mounting, we may take the outflow loss as another characteristic constant and express it as a percentage of the total head

$$L = \frac{u^2}{2gH} \quad \ldots\ldots\ldots\ldots \quad (1)$$

Let the peripheral velocity of the tips of the wheel vanes equal

$$\phi\sqrt{2\,g\,H} = \pi\,D_1 \times \frac{s}{60} \quad \ldots\ldots\ldots \quad (2)$$

where ϕ is a constant characteristic of a given wheel.

Equation for the power of the runner:

$$P_e = \frac{62.4\,Q\,H\,\varepsilon}{550} \quad \ldots\ldots\ldots \quad (3)$$

where

$$Q = \frac{\pi}{4} f^2 u D_1^2$$

$$D_1 = 60 \phi \frac{\sqrt{2gH}}{\pi s} \quad \ldots\ldots\ldots \text{ from (2)}$$

$$u = \sqrt{2gLH} \quad \ldots\ldots\ldots \text{ from (1)}$$

Substituting these values in equation (3), we have, assuming value of $\phi = 0.7$ and simplifying:

$$s = \sqrt{\frac{8216 f^2 \varepsilon H^{5/2} \sqrt{L}}{P_e}}.$$

Transposition of the Formula to the well-known Formula for Specific Speed.

For a given design of runner, but built in various sizes all geometrically similar to each other, we shall have ϕ, f, L and ε all constant when any one of these runners works at the best speed corresponding to any head. Therefore, for this particular design of runner, we can put,

$$s = c^1 \cdot \frac{H^{5/4}}{\sqrt{P_e}}.$$

If one of the wheels of this set be built of such a size that when operating under a head of unity (1 meter in metric system) it will deliver 1 HP, then c^1 is seen to be the speed of this wheel when operating under 1 meter head. $c^1 = s \frac{\sqrt{P_e}}{H^{5/4}}$ Call this the specific speed and denote it by N_s; then the speed of any wheel of the set, when working at its proper speed under any head, will be

$$s = N_s \cdot \frac{H^{5/4}}{\sqrt{P_e}}$$

where N_s is a constant, typical of the series of the designs. For any given runner, the specific speed may be found from the formula

$$N_s = s \frac{\sqrt{P_e}}{H^{5/4}}$$

and if N_s is desired in the metric system, P_e and H must be taken in that system.

The value of ϕ, or the ratio of the peripheral velocity at the mean diameter of the runner inflow to the spouting velocity, may vary anywhere from 0.65 for very high head wheels, to 0.82 for very low head wheels.

99. Water Wheels of the Mc Call Ferry Power Company.

These wheels, fig. 153, are each capable of developing 13500 HP, when operated at 53 feet head, and with 80% gate opening. Each one is coupled to a 7500 kilowatt three-phase 25 cycles electric generator. The speed is 94 revolutions per minute. The design secures that the wheels will be capable of giving full output with 100% gate opening when the available head is reduced to the possible minimum during extreme floods. It will be noticed that there are two wheels of the inward and down-

ward flow type mounted on a single shaft. The shaft is of forged steel and is in one piece from the bottom to where it is coupled to the generator shaft. Its weight and that of all the parts borne by it, including the revolving field of the generator, are carried on a thrust bearing, which is itself supported on a lens-shaped casting set into the masonry. The latter casting is of such size as, when removed, to permit withdrawing the shaft with turbine runner attached. The runners are cast of iron in one piece and are carefully finished by hand so as to present smooth curved surfaces to the water. Each is about ten feet in diameter. The arrangement for operating the gates consists of a vertical shaft made of pipe, carried on a step and held at the top by a bearing attached to the wall. This shaft which is connected to both the rings which carry the gates of the two turbines, is revolved in one direction or the other, under the control of the speed governor.

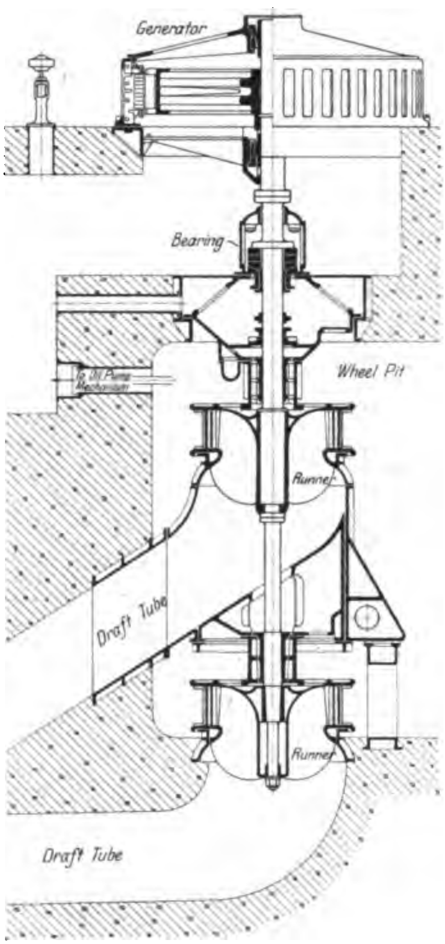


Fig. 153. 13,500 HP Water Wheel of the McCall Ferry Power Co.

The water wheel data is as follows:

Horse Power (80 % gate opening)	13,500
Number of runners	2
Speed	94 r. p. m.
Head	53 fe et
Volume of water at full head, per second	2800 cu. feet
Total weight on thrust bearing	335000 lbs.
Total weight of metal in 1 turbine . . .	650000 lbs.

Francis type wheel, inward and downward flow.
Efficiency, full load, 80 % or better.

These wheels were built by the I. P. Morris Company of Philadelphia, and the design was the object of a valuable method of graphical analysis developed by Mr. H. B. Taylor, under the supervision of W. M. White, and given, in bulletin No. 2 of the I. P. Morris Co.

Curve No. 1, fig. 154 shows the power which the wheel will give for heads varying from 70 feet to zero, provided that the revolutions are allowed to vary as the square root of the head. The horse-power of a given wheel will vary as the three halves power of the head and the efficiency will remain constant, provided the revolutions are also permitted to vary to best suit the conditions of head. Referring to fig. 154,

it will be noted from curve No. 1 that at the 70 ft. head mark, the wheel would develop 15,000 horse-power, and from curve No. 6 it will be noted that the best speed for the wheel under the conditions of 70 ft. head would be 111 revolutions per minute. It will also be noted from curve No. 1 that under the 50 ft. head the wheel would develop 9150 horse-power, if it be run at 94 revolutions per minute. By keeping a constant ratio between the periphery of the runner and the square root of the head, the efficiency of the wheel at varying heads is not changed for any given setting of the gate. This ratio, which we have denoted by Φ, is usually about 0.7 for reaction wheels. In order to properly utilize the output of the wheel it is necessary that the speed be kept constant.

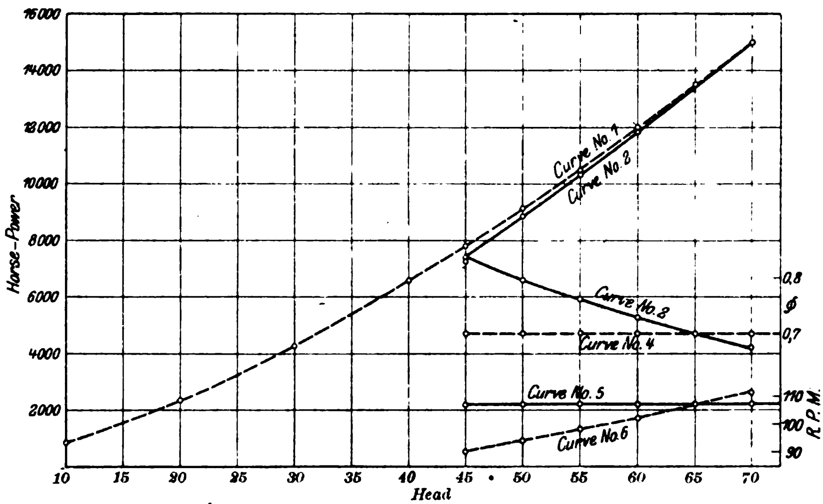


Fig. 154. Wheel designed for 13,500 HP, 65 Ft. Head, 107 r. p. m.

Power-Head Curve No. 1. Φ Constant, R. P. M. Variable.
Power-Head Curve No. 2. Φ Variable, R. P. M. Constant. Corrected according to Shawinigan test for loss in HP due to speed not best for the given head.
Φ-Head-Curves Nos 3. and 4.
Φ is a relation between periphery of runner and head.
Head-R. P. M.-Curves Nos. 5 and 6.

In order to determine the amount of power that will be lost by keeping the speed constant while the head varies, the curves on fig. 155 were plotted from actual observations. Curve No. 1 is the full gate reading of the 10,500 horse power turbine which was installed for the Shawinigan Water and Power Company. This wheel was designed for 10,500 horse-power when working under a head of 135 feet and when running at 180 revolutions per minute. The observations which are plotted on this curve were obtained by using the generator as a brake for the wheel, and a water rheostat was used as a means of loading the generator. The speed was then adjusted to 180 revolutions per minute at the wide open gate and an observation made. By varying the field of the generator, the speed of the unit was varied without materially affecting the power, and without moving the gate of the wheel. Observations were made above and below the normal speed through as wide limits as the rheostat in the field circuit of the generator would permit. The power output was determined by

means of accurately calibrated electrical instruments. The speed was determined by an accurately calibrated tachometer. The curves on fig. 155 give the relation between Φ and horse-power. Referring back to fig. 154, and taking the 50 feet head conditions, it is noted that for a constant speed of 107 revolutions per minute, Φ would have to increase to 0.8. By referring again to fig. 155, it is noted that when Φ was 0.8 with full gate opening, the power dropped from 10 600 HP to 10 250 HP, or about 3.3 %. Reducing the power at the 50 feet head on fig. 154, (curve No. 1) by 3.3 %, we obtain a point on curve No. 2 which gives the actual power which would be developed by a wheel under the 50 ft. head, and running at the constant speed of 107 revolutions per minute. Curve No. 2 is plotted in this manner from curve No. 1. As a check to curve No. 1 in fig. 155, curves Nos. 5, 6, 7, and 8 have been plotted, all of which were made from actual observations, in the same manner as curve No. 1. All of these wheels are Francis inflow and were designed for Φ equal to 0.7, except curve No. 6, which is designed for Φ equal to 0.5; No. 6 is an outward flow Fourneyron wheel. Curve No. 5 is for a 6000 horse power wheel with gates in the draft tubes. The shape of the curve shows that the gate was probably not entirely open when the observations were made. In fig. 156, there are plotted efficiency curves which the designed wheel would give under varying heads and running at a constant number of revolutions. Curve No. 1 is an exact duplicate of the efficiency curve obtained on a

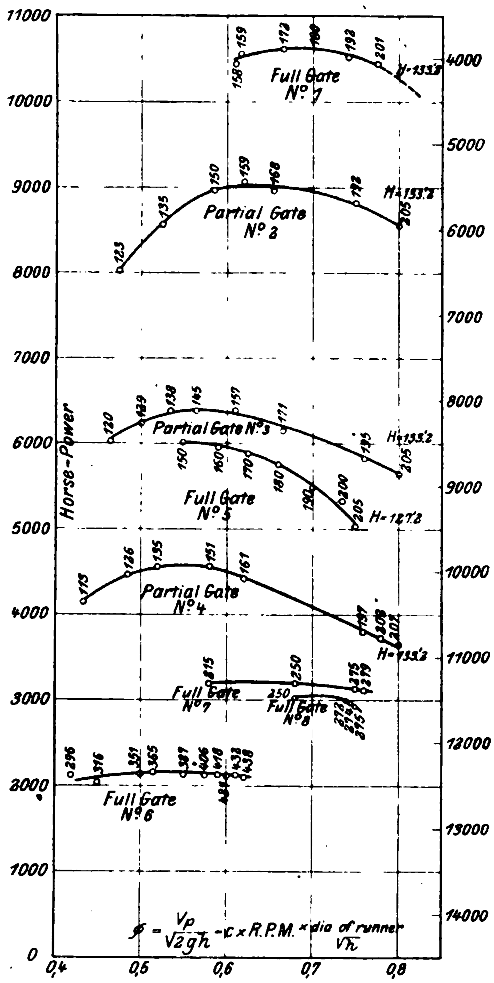


Fig. 155. ϕ-Horse-Power Curves.

3500 HP wheel working under 210 feet head and making 250 revolutions per minute. The test of this wheel was made by the engineers of the Niagara Falls Hydraulic Power & Manufacturing Company. The observations were carefully made, and the results accurately determined. The water flowing from the wheel was determined by means of a weir constructed in the tail race. The weir was arranged with end contractions, and the Francis formula was used in determining the quantity of water flowing. The power output of the wheel was determined by using the generators as a brake on the wheels. The power output from the gene-

rators was determined by accurately calibrated electrical instruments. The summation of these readings divided by the efficiency of the generator gave the power output at the flange coupling of the turbine shaft. The factory tests of the efficiencies of the generators were taken as correct. The wheel is of the Francis inflow type, with double runners, fitted with movable guide vanes, similar to those proposed for use in the wheels for the Mc Call Ferry Power Company. It will be noted that the efficiency of the wheel reaches 82.3% at about seven eighths power, the efficiency

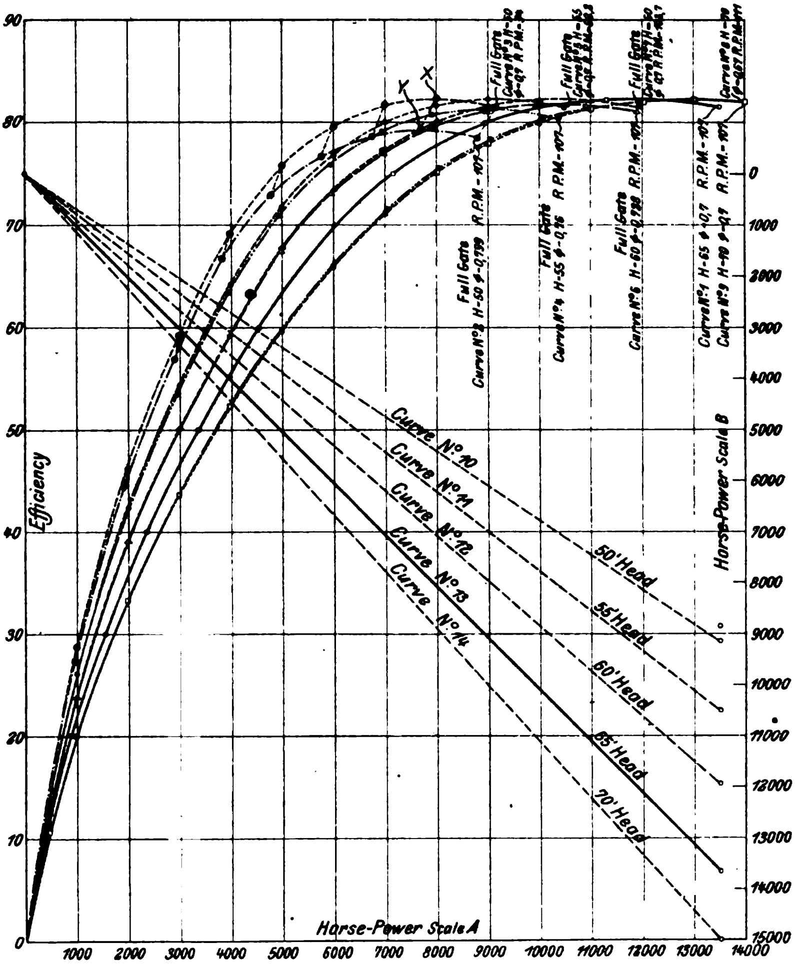


Fig. 156. Efficiency Curves

dropping to 81.5% at the wide open gate. It will be noted that the efficiency is very high at part load. This was accomplished in the design of the wheel by sacrificing the maximum efficiency which could be reached in order to obtain a higher efficiency at part loads. This curve is taken as typical of the efficiency which would be obtained by the wheel proposed for the McCall Ferry Power Company, when working under a 65 ft. head. The efficiency curve of the 10,500 horse power wheel supplied to the Shawinigan Water & Power Company gives higher results than

the curve selected. but it is believed curve No. 1 is the best for a typical curve. Curve No. 1 was plotted on Fig. 156 by assuming that at wide open gate 3500 horse power corresponded to 13,500 horse power in the designed wheel. The part gate points of the curve were obtained by proportion. Curve No. 3 represents the efficiency and power of the wheel when working under 50 ft. head and at 94 revolutions per minute. Point *A* on this curve is obtained in the following manner: First, read on curve 1, fig. 154, the power which the wheel would give under 50 ft. head, and revolutions best suited. This is found to be 9150 horse power. On scale *B*, fig. 156, draw a line from 9150 HP to zero, forming curve 10. To find what the efficiency would be at 8000 horse power under the 50 ft. head, take the point at 8000 horse power on scale *B*, projected horizontally until it intersects curve 10, and read 11,800 horse power. This point is projected vertically until it is over 8000 horse power on scale *A*. Thus the point *A* on curve 3, is obtained which reads the efficiency of the wheel when developing 8000 horse power under the 50 ft. head and running at the revolutions best suited, namely 94. It is desired to run this wheel, however, at 107 revolutions per minute, under all conditions of head, and it is necessary to correct curve 3 for the drop in power and efficiency due to increase in speed. Referring to curve 1, fig. 154, it is noted that the power varies, and in the calculations of efficiency on fig. 156 it has been supposed that the efficiency varies directly as the power. In other words, it is supposed that the quantity of water does not vary when the revolutions are changed with the constant setting of the gate. This is not strictly true, however, but for the observations as plotted on curve 1, fig. 155, the quantity of water would vary probably one half of one per cent, increasing as the revolutions decrease, and decreasing as the revolutions increase (from 158 to 201). Referring to fig. 154 and the 50 ft. head, it is noted that when the speed is increased from the best speed of 94 revolutions to the desired speed of 107 revolutions, the power falls 3.3 %, and the power and efficiency of the points on curve 3 are decreased 3.3 %, resulting in curve 2, which gives the desired curve for the operation of the wheel under 50 ft. head and 107 revolutions per minute. Referring to fig. 155, curves 1, 2, 3 and 4, it will be noted that the slope of these curves between Φ equaling 0.7 and Φ equaling 0.8, is about the same, and it is possible, therefore, to reduce the power and efficiency of the points on curve 3 by the same percentage, namely 3.3 %. In this manner curve 2 is obtained, which gives the power and efficiency of the wheel when working under the 50 ft. head, and running at the speed of 107 revolutions per minute. In this same manner curves 5 and 7 are plotted, curves 4 and 6 being deduced therefrom respectively. In the same manner curve 9 is plotted and curve 8 deduced therefrom. It will be noted that curve 8 lies on the opposite side of the parent curve to that of the other curves. Curve 8 crosses curve 9 at 13500 horse power on scale *A*, and beyond this point would drop below curve 9. The reason curve 8 falls to the left of curve 9, and shows greater efficiency at part gate for the 70 ft. head is because when Φ changes from 0.7 to 0.65 (in fig. 155), the partial gate curves 2, 3 und 4, fig. 155 show the increase in power and efficiency. These points, however cannot be very definitely

determined, but it does show that it is right to assume that the designed wheel, when working under the head of 70 ft. and running at 107 revolutions, will show higher percentage of efficiency at part gates, than when running at the 65 ft. head and the same powers. The curves in fig. 156 show that the efficiency is not seriously affected by keeping the speed of the wheel constant under the varying conditions of head. They do show however, that the power is seriously affected under the varying head. The endings of the various curves show the maximum power, as read on scale A,

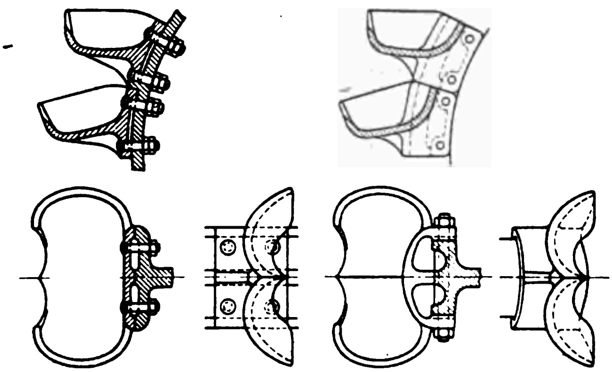


Fig. 157. Pelton Buckets and Methods of Securing same to Rim.

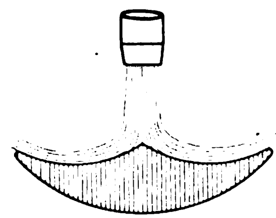


Fig. 158.

which the wheels will give under that head. These curves, therefore, give the performance of the wheel when running at a constant number of revolutions and working under varying heads from 50 to 70 feet. The curves, of course are not absolutely correct. They show however, fairly accurately, the amount of variation in efficiency and power which may be expected from the actual conditions obtained with the proposed wheel under the head for which it was designed.

100. Impulse Wheels,

Pelton or impulse wheels may be considered as radial turbines, operating under water pressure applied to the outer periphery. The velocity of the jet issuing from the nozzle is

$$V = 0.95\sqrt{2gH} \quad \text{to} \quad 0.98\sqrt{2gH}.$$

In order to obtain a discharge at right angles to the circumference of the wheel, the peripheral velocity varies from

$$0.45\sqrt{2gH} \quad \text{to} \quad 0.50\sqrt{2gH}.$$

Calling

s = the speed in r. p. m., and
D = the diameter in feet.

we have the following relation

$$\frac{\pi D s}{60} = 0.45 \quad \text{to} \quad 0.50\sqrt{2gH}.$$

The diameter of a circular nozzle has the following relation to the diameter of the wheel[1]

$$d = 1.2\,D - 0.0366\,D^2 + 0.2.$$

[1] Gelpke—Van Cleve. Hydraulic Turbines.

The quantity of water in cu. ft. per sec. flowing from each nozzle is

$$Q_1 = \frac{\pi d^2}{576} k \sqrt{2gH}$$

where

$$k = 0.95 \text{ to } 0.96.$$

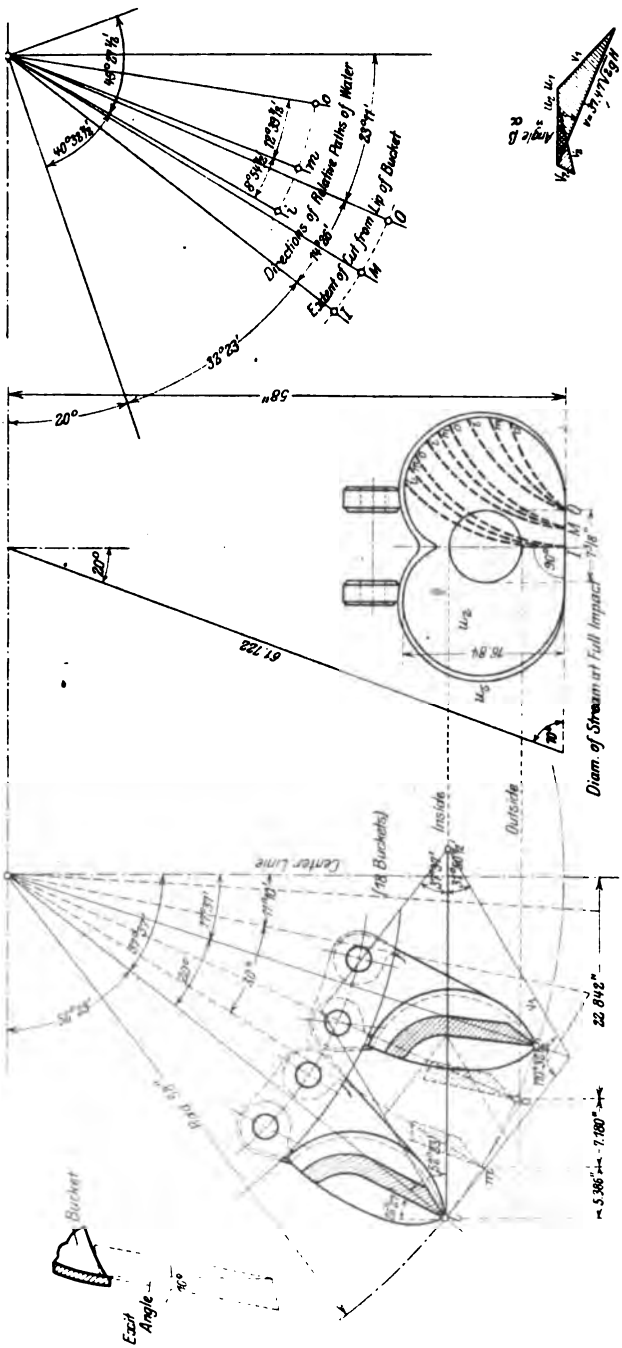


Diagram A. Impact on Rim of Bucket I. M. O. Diagram B. Diagram C. Plan of Diagrams.

Diagram for Determination of Circumference Velocities u_1 and u_2.

Fig. 159.

H = Total Fall = 865 Ft.

u = Wheel Velocity at Point of Entrance } Entrance with Index 1 } Diagram C.
V = Absolute Velocity of Water } Exit " " 2 }
v = Relative " "

α = Angle between u_1 and V_1.
β = " " u_2 " v_2.
$u_2 = 21.111\sqrt{2gH}$ – Circumference. Velocity of Outer Point of Bucket.

ψ = Angle Corresponding to α.
Friction Loss in Bucket of 180 $\frac{v_2^2}{2g}$.
S_n = Loss in Needle Tip.

S_r = Friction and Foam Loss in Bucket.
S_s = Impact Loss at Entry of Jet into Bucket.
S_a = Exit Loss in Jet from Bucket.
S = Total Losses in Per Cent.

If n is the number of nozzles, then:

$$n = \frac{Q}{Q_1}.$$

In the case of Pelton wheels. the nozzles are generally made circular or square in cross section. The runner is formed by a cycle of cups either cast on a rim or secured to it. Some forms of cups or buckets are illustrated in fig. 157; and the effect of the water jet on such a bucket is illustrated in fig. 158. Fig. 159 shows the diagrams used in designing the wheel-buckets of the Kern River Plant No. 1. These diagrams were developed by F. C. Finkle and are of interest to the designer. Impulse wheels do not operate satisfactorily if submerged. and really should always be located above tail water.

101. Type Characteristics of Turbines.

In cases of medium heads, as from 300 to 500 feet, the most suitable type of turbine cannot readily be determined. In order to facilitate this determination, however, a factor known as the type characteristic of a turbine has been developed. and is represented by the expression:

$$K_t = \frac{s}{H}\sqrt{\frac{P_e}{\sqrt{H}}}$$

in which

K_t = type characteristic.
s = speed of turbine in r. p. m.,
H = effective head in feet, and
P_e = capacity in horse power of turbo-unit.

The factor K_t is composed of

1st a power constant

$$k_1 = \frac{P_e}{QH}$$

which is the power of a similar runner using 1 cu. ft. of water under a one foot head;

2nd a capacity constant

$$k_2 = \frac{Q}{D^2\sqrt{H}}$$

which is the discharge of a similar runner of 1 foot diameter under a head of one foot; and

3rd a speed constant

$$k_3 = \frac{\pi D s}{60\sqrt{H}}$$

which is the speed of the runner under a head of one foot.

Then

$$K_t = \frac{60\, k_3 \sqrt{k_1\, k_2}}{\pi}$$
$$= \frac{s}{H}\sqrt{\frac{P_e}{\sqrt{H}}}.$$

Experience shows that a radial inward flow turbine gives a high type characteristic varying between 10 and 100, whereas the type characteristic of an impulse wheel is reduced to between 1 and 5. If an inward flow turbine has more than one runner, the *HP* to be applied in the above formula is the power developed by one runner only. just as in an impulse wheel, the power developed by one nozzle only is to be used. The following table will enable the designer to choose the proper type of turbine for specified cases.

Table XLI.

Type of radial inward flow turbine	K_t Type characteristic
Low speed turbine	10 to 20
Medium speed turbine	30 to 50
High speed turbine	60 to 80
Very high speed turbine . . .	90 to 100

If K_t is greater than 100, multiplex turbines must be used. ***Example;*** suppose a 1250 *HP* unit is to be run at 300 r. p. m., with an effective head of 360 feet; the type characteristic is found to be

$$K_t = \frac{300}{360}\sqrt{\frac{1250}{\sqrt{360}}} = 6.7$$

in which case an impulse wheel would be used. If the wheel capacity were 3600 *HP* under the same head and running at 500 r. p. m.,

$$K_t = \frac{500}{360}\sqrt{\frac{3600}{\sqrt{360}}} = 19.2$$

in which case a radial inward flow turbine would be used.

102. Turbine Efficiency.

The efficiency of a turbine may be obtained by a curve such as shown in fig. 160. Gelpke determines ε as a function of the size of the unit and gives the following values:

$\varepsilon \approx 78\%$ for a turbine unit of 30 HP
$\varepsilon \approx 87\%$ for a turbine unit of 10,000 HP.

Intermediate values may be obtained from the curve and necessary deductions must be made for friction losses in step and guide bearings and for hydraulic losses in balancing pistons and cooling devices; these losses may be taken as follows:

for a hand regulated

30 HP unit,	3%
10,000 HP unit,	$1\frac{1}{2}\%$

for automatic regulation of a

30 HP unit,	1%
10,000 HP unit,	$\frac{1}{2}\%$

These losses must be allowed for the power expended in regulation. The hydraulic losses besides those occuring within the turbine are to be distributed as follows in percentage of the total pressure head[1]

$l_1 \approx 3\,^0/_0$, in entering the wheel pit,

$l_2 \approx 3$ to $5\,^0/_0$ occurring in the draft tube and depending upon its form and length, and

$l_3 \approx 0$ to $1\,^0/_0$ if the discharge from the draft tube is perpendicular to the direction of the tail race.

This last loss can be entirely obviated by the use of a draft tube the direction of which is gradually changed to that of the water in the tail race. In practice, it is best to assume

$$(l_2 + l_3) = 5\,^0/_0.$$

It is also to be noted that there should be no sudden changes in the velocity or direction of flow at the point where the water passes from the runner to the draft tube.

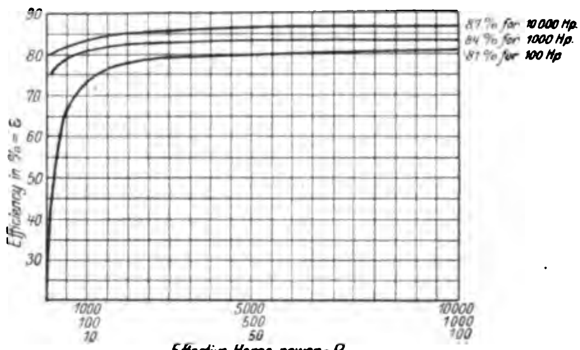


Fig. 160.

103. The Testing of Water Wheels.

There are several methods in use for the testing of water wheels, of which the most important are:

1. The Prony brake method (fig. 161),
2. the chemical method, and
3. the indirect method.

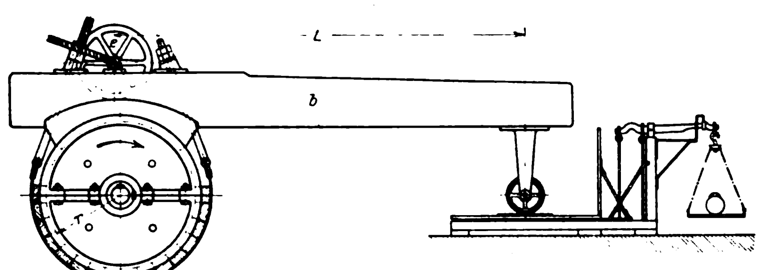


Fig. 161.

The latter is employed mainly for the testing of wheels in place, a problem that is calculated and resolved by means of the electric current readings at the switchboard, knowing the efficiency of all intermediary apparatus. A detailed measurement by this method is given in the following paragraph. (§ 106.)

[1] Victor Gelpke. Turbinen und Turbinenanlagen.

The Prony brake method consists in the use of a lever b, clamped on the pulley of the wheel to be tested, said pulley being surrounded by a wood-lined jacket fixed to the lever system, as shown in fig. 161, cooled and lubricated by a stream of soap water. The pull of the pulley on the jacket is weighed, the friction being regulated at the temper screw, e, so that the weights are kept balanced. When such equilibrium is obtained, note is taken of the number of revolutions per minute, and the formula applied:

$$P_e = \frac{W s l C}{33,000}$$

in which

P_e = horse power of wheel,
W = net weight in lbs. on lever arms,
s = revolutions per minute,
C = circumference of brake, and
l = ratio of lever arms.

At the same time, the theoretical power of the water is to be computed by the formula:

$$P_t = \frac{Q H W_t}{550}$$

in which:

Q = cubic feet per second passing the wheel,
H = head of water on wheel, in feet, and
W_t = weight in lbs. of 1 cu. ft. of water, according to temperature.

Having calculated the theoretical horse power of the water, and determined the effective power of the wheel by means of the Prony brake, as indicated above, the efficiency of the wheel is found by the ratio:

$$\text{Efficiency of wheel} = \frac{P_e \text{(wheel)}}{P_t \text{(water)}}$$

For the purpose of making the necessary experiments on water wheels, the Holyoke Water Power Company built a permanent testing flume, in which wheels are tested both for power and amount of water discharged. The discharge is measured by a weir, the crest of which is formed by a piece of planed wrought iron, and which can be used with or without end contractions. The depth of water on the weir is measured by a hook gage. The quantity of water passing the weir is computed by the well known formula:

$$Q = 3.33\,[L - 0.10\,n H]\,H^{3/2}$$

in which:

Q = quantity of water in cu. ft. per second,
L = length of weir in feet,
n = number of end contractions, and
H = depth of water on the weir, in feet.

If the volume of water makes it necessary, Q is corrected for velocity of approach.

According to M. A. Blanchet, it results from the use of this formula:

1. For a weir without end contractions (see page 31):
   a) that the discharges so calculated are smaller than the real discharges, the Bazin formula (see page 31):

$$Q = \mu \left[1 + 0.55 \left(\frac{D}{D+P}\right)^2\right] LD\sqrt{2gD}$$

   being of more accuracy for such measurements;
   b) the efficiency of American wheels measured by means of the Francis formula for discharge measurements, are in favor of a better efficiency for American wheels, all other things remaining the same.
2. For a weir with end contractions:

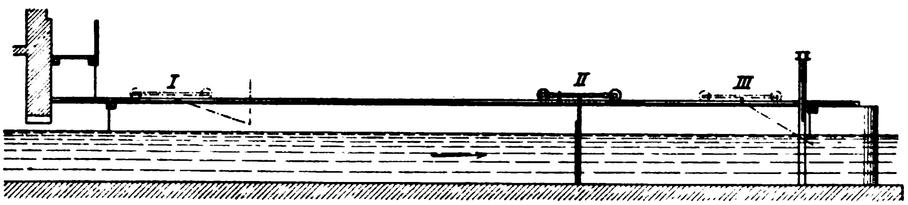


Fig. 162. Arrangement for Measuring Tailrace Water in Testing Turbine by the Curtain method.

In this case also, but in a smaller degree, the use of the Francis formula shows a better efficiency.

The following table gives comparative results of the Holyoke tests of a 16 inch turbine, the same wheel being afterward tested in Germany by Prof. Pfarr:[1]

Discharge	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3
Holyoke tests	0.81	0.795	0.765	0.725	0.67	—	—	—
German tests	0.718	0.703	0.693	0.658	0.591	0.491	0.358	0.121

Prof. Erik Anderson has developed another method for measuring the discharge of turbines. A tail race of uniform cross section and of smooth surfaces has been built; a smooth track runs along it, over which a light carriage made of aluminum is moved. A light frame work, either of wood or of steel, is hinged to the carriage, its width being equal to the width of the tail race, providing a clearance of $\frac{1}{4}$ to $\frac{3}{8}$ inch between the frame work and the sides of the tail race. This framework is covered with a water proof canvas. The general layout of the testing arrangement is shown in figs 162 and 163. At the point I, the curtain is lowered and soon assumes a vertical position before entering the area of measurement, its working position being shown at point II, and the point III shows the carriage with the curtain released from the vertical position, after which it is drawn back for another run. The speed of the carriage depends on the average velocity of water through the tail race channel. The position and time of travel are recorded by electrical contacts placed from three to five feet apart.

[1] Zeitschrift des Vereines deutscher Ingenieure. June 7—1902.

The *chemical method* consists in placing a determined amount of soluble salts into a reservoir containing an unknown amount of water.

Call: W = weight of unknown amount of water,

p = known weight of concentrated solution of salts, and

$\frac{m}{n}$ = the degree of concentration of this solution.

Then the amount p of solution contains:

$$\frac{p_m}{m+n} \text{ of salt and } \frac{p_n}{m+n} \text{ of water.}$$

This quantity of salt diluted in an unknown quantity of water, will give a solution of concentration $\frac{a}{b}$ which may be determined by analysis. Knowing $\frac{a}{b}$, W is easily found by the relation

$$W = \frac{p}{m+n}\left[\frac{mb}{a} - n\right] \text{kg.}$$

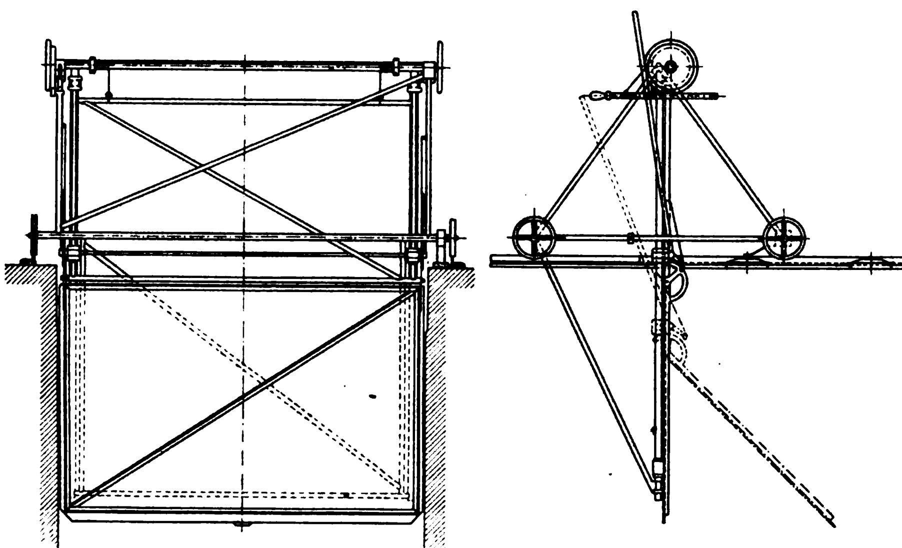


Fig. 163. Carriage with Curtain for Testing the Water Discharge of Turbines.

Therefore, injecting such a concentrated solution at equal intervals of time (each second) into the pipe leading to the turbine, and analyzing the water that may be collected at the exit of the turbine, the amount of water may be calculated. Dr. R. Mellet has tested the discharge of a turbine by this method, using neutral solutions of alkaline chlorides, the latter being tested by silver nitrates and potassium chromate. The addition of silver salts to a chloride precipitates silver chloride, the color of which is white, and the liquid takes a tint of lemon yellow. When the reaction is terminated, the excess of the silver solution forms a precipitate, reddish brown in color, of silver chromate, so that the original lemon yellow color takes an orange tint. This excess may be noted as a function of the number of cubic centimeters used, and written $\frac{1}{p}$.

Let: N_1 = number of cubic cm. of centi-normal solution, necessary to titrate 1 liter of initial solution,

n = number of cubic cm. necessary to titrate 1 liter of water from the turbine, taken before the test, and

N_2 = number of cubic cm. to titrate 1 liter of final solution.

Deducting the constant fraction, the number of cubic cm. becomes:

$N_1 - \frac{N_1}{p}$ cubic cm. for 1 liter of the initial solution,

$n - \frac{n}{p}$ cubic cm. for 1 liter of the incoming water, and

$N_2 - \frac{N_2}{p}$ cubic cm. for 1 liter of the final solution.

The equivalent of sodium chloride being 58.5, each cubic cm. of the centi-normal solution indicates 0.000585 gram of salt. Consequently:

the initial solution contains $0.000585\left[N_1 - \frac{N_1}{p}\right]$ grams of salt per liter,

the feeding water contains $0.000585\left[n - \frac{n}{p}\right]$ grams of salt per liter,

the final solution contains $0.000585\left[N_2 - \frac{N_2}{p}\right]$ grams of salt per liter.

The discharges are inversely proportional to the concentrations, therefore, calling x the discharge of the final solution, and d the discharge of the initial solution, we have

$$x = d\frac{0.000585\left[N_1 - \frac{N_1}{p}\right]}{0.000585\left[N_2 - \frac{N_2}{p}\right] - 0.000585\left[n - \frac{n}{p}\right]} = d\frac{N_1}{N_2 - n}$$

and the required discharge of the turbine would then be denoted by:

$$Q = d\left[\frac{N_1}{N_2 - n} - 1\right].$$

The test was accomplished as follows:

The initial solution was injected through an opening at the turbine entrance, the discharge of this solution being 0.1211 liter per second. A sample of this solution as well as of the running water had been taken before the test. During the test, samples were taken after six and nine minutes.

1. Test of the initial solution: 10 cu. cm. were taken, which were then diluted in 1 liter, and 10 cubic cm. of this liter were analyzed. For this fraction, 35.5 cubic cm. of the silver solution were necessary, therefore $N_1 = 355\,000$.

2. Test of the natural water. One liter was evaporated. After reducing the volume to about 10 cubic centimeters, 3.8 cu cm. of silver solution were necessary, therefore $n = 3.8$.

3. Test of final solution. Sample taken at the end of six minutes. The supposed discharge was taken at 100 to 300 liters per second. Therefore from 100 to 300 cu cm. had to be evaporated and 33.5 cu cm. of the nitrate of silver solution were necessary in the 200 cu cm. which were

actually evaporated; this amounts to a quantity of 167.50 cu cm. per liter. Therefore $N_2 = 167.50$.

The test of the sample taken at the end of nine minutes gave the value $N_2 = 167.67$.

The discharge was then found by applying the formula, giving

for sample (1) $Q = 0.1211\left[\frac{355\,000}{167.5 - 3.8} - 1\right] = 262.4965$ liters,

for sample (2) $Q = 0.1211\left[\frac{355\,000}{167.67 - 3.8} - 1\right] = 262.2241$ liters

resulting an average of 262.36 liters per second, and a difference between the two samples of $0.104\,^0/_0$ demonstrating the great accuracy of this method.

104. Schaghticoke Turbine Tests.

The hydroelectric system being worked at different load factors, it is required to determine the power and efficiency of the turbine, said power and efficiency to correspond to the readings made at the switchboard, supposing known the power and efficiency of the electrical apparatus. The difficulty consists in finding the exact head. In effect when the turbine is running, the measure of the head is as indicated in fig. 164, and is composed of three parts:

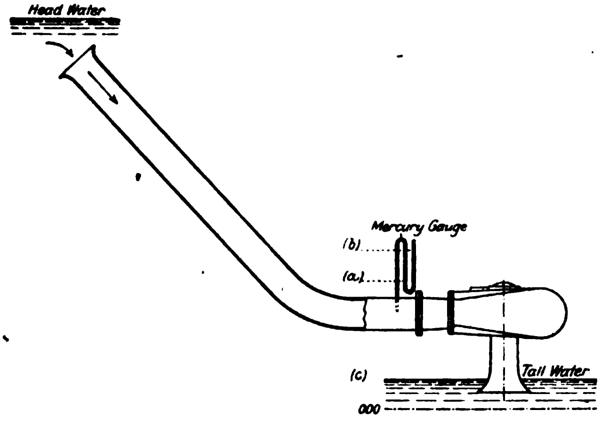


Fig. 164.

1st The head corresponding to the hydrostatic pressure on the center of the pipe at the turbine entrance, this head being readily measured by means of a mercury gauge.

2nd The head corresponding to the difference of elevation between the center, of the pipe at the turbine entrance and the level of tail race water.

3rd The head corresponding to the velocity of the water in the pipe.

The total effective head is therefore given by the relation:

$$H_{eff} = [b - a]\,\gamma + [a - c] + h_v$$

in which

γ = density of mercury = 13.58,
h_v = velocity head.

The first two parts of this expression are easily determined, but the measurement of h_v offers certain difficulties, which are eliminated however, by the following method. The head corresponding to a certain velocity is found by the expression:

$$v = \sqrt{2gh}$$

and may be determined by means of a Pitot tube, in the following conditions: Two grooved traverses are fixed inside the pipe, about 150 feet distant from the power house. One of these traverses corresponds to a vertical diameter, and the other to a horizontal diameter. Each one receives a Pitot tube in its groove, sliding at will, and connected to a glass tube located on the board shown in fig. 165. These Pitot tubes can occupy 15

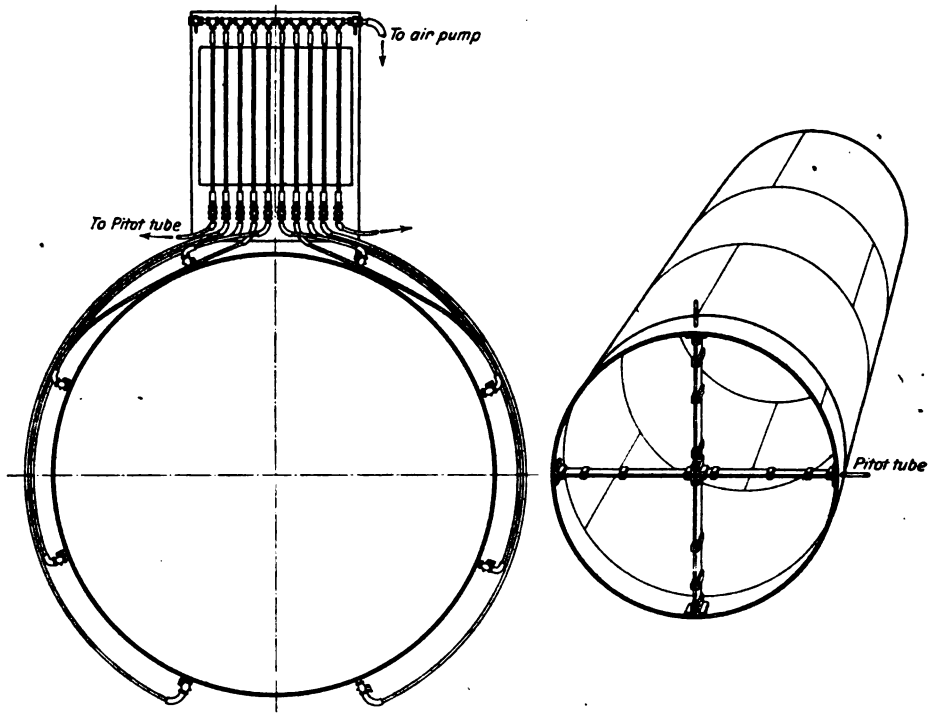


Fig. 165. Arrangement of Pitot and Static Tubes.

different positions on each traverse. On the board, a graduated scale is mounted, together with ten glass tubes, of which eight are connected to their respective cocks, equally distributed along the circumference of the pipe. These serve to indicate the static pressure; the extreme glass tubes are connected to the Pitot tubes. If the cocks are opened, the water will rise inside the tubes, and its level its maintained inside the limits of the graduated scale by means of compressed air. Herewith is given in detail the work done to determine the efficiency of the 5000 HP turbines of the Schaghticoke Plant at $^3/_4$ load. The diameter being variable on account of the superposition of the plates, an average diameter of 6.014 ft. has been taken, giving a pipe area of 28.37 sq.ft. Table XLII shows the distance from the center of the pipe, of the 15 positions occupied by the Pitot tubes on the traverses. It is clear that to take readings of each static tube for the 15 positions of the Pitot tubes would require 30 $\times$ 8 operations, and would complicate the work, introducing thereby possibilities of error. It is better therefore, to gauge successively all the static tubes with the Pitot tubes, and establish a correction factor which will permit the making of

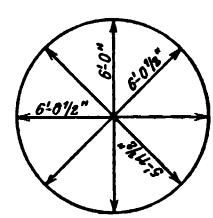


Fig. 166.

a single reading at the Pitot and its adjacent static tube. The gauging of the two Pitot tubes is effected by maintaining at zero the level of the adjacent static tube. Readings of the Pitot and of their respective adjacent static tubes are then taken for the 15 positions of the Pitot across the vertical diameter, making three tests in each position. The factor of correction is then applied to the average of the three tests. These results permit the calculation of the velocity by means of the formula:

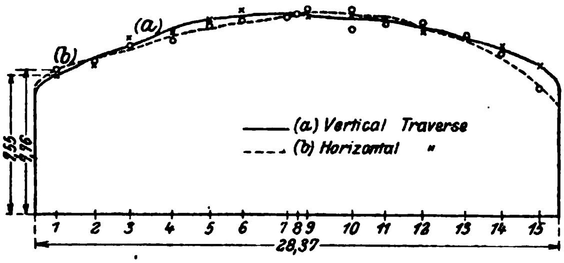


Fig. 167. Discharge Curves.

$$v = \sqrt{2gh}.$$

Tables XLIII and XLV correspond to tests relative to the vertical traverse. The same operation is applied to the horizontal traverse, finding results as per tables XLIV and XLVI. The observations corresponding to the hydrostatic pressure $(b - a)$ have given:

1st test	9.684 ft.
2nd "	9.693 "
Average	9.688 ft.

The value of $(a - c)$ was found to be:

1st test	10.36 ft.
2nd "	10.32 "
Average	10.34 ft.

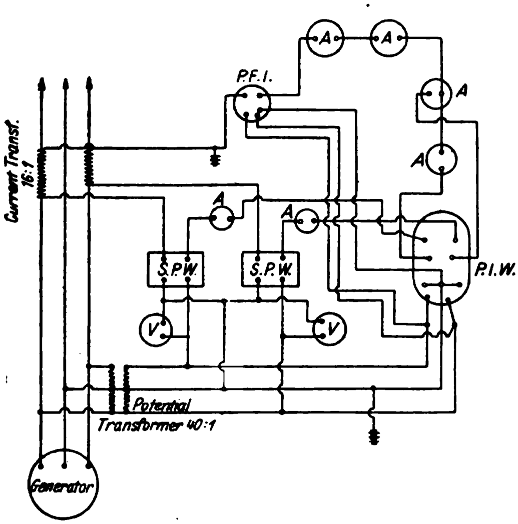


Fig. 168. Switchboard Connections.

Having calculated the velocities as indicated above, and it being necessary to calculate the actual discharge, a curve is plotted as per fig. 167, in which the abscissae are proportional, not to the lengths corresponding to the 15 divisions on the diameters, but to the corresponding areas, which have been given in table XLII. The ordinates corresponding to the velocities are given in the last column of tables XLV and XLVI. The total length of the base of fig. 167 represents the area of the pipe section, and the area of the curve, being measured with a planimeter, gives the actual discharge per second. In the case under consideration, the respective areas correspond to the following discharges:

Curve *a*, fig. 167, vertical traverse	. .	276.45 cu. ft.
Curve *b*, fig. 167, horizont. traverse	. .	270.71 cu. ft.
	Average	273.58 cu. ft.

The area of the pipe at the turbine entrance is 17.104 sq. ft., and the calculated discharge 273.58 cu. ft. per sec. Therefore the average velocity of the water at the turbine is

$$\frac{273.58}{17.104} = 15.995 \text{ ft. per sec.}$$

The head corresponding to that velocity is

$$\frac{(15.955)^2}{64.4} = 3.973 \text{ ft.}$$

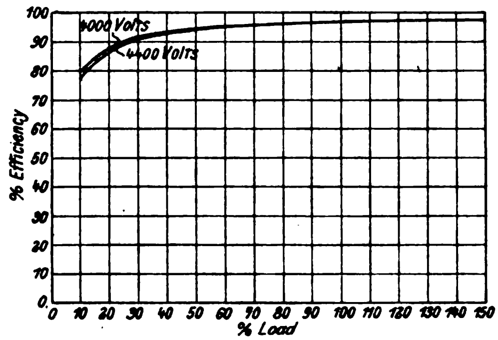


Fig. 169. Commercial Efficiency of a 3000 kw Generator.

The static pressure given by the mercury gauge is

$$9.688 \times 13.58 = 131.563 \text{ ft.}$$

The total effective head is therefore

	131.563 ft.
	10.340 „
	3.973 „
Total:	145.876 ft.

The total effective head being determined as above, the actual power of the turbine is calculated by considering the power and efficiency of the generator and electrical apparatus. The connections of the electrical machinery is given on fig. 168, and the efficiency of the alternators on fig. 169.

The mean power of turbine is 3866.5 *HP*, (table XLVII).

The theoretical HP developed by 145.876 ft. head is:

$$\frac{273.58 \times 145.876 \times 62.5}{550} = 4535\ HP.$$

So the efficiency is

$$\frac{3866.5}{4535} = 85.26\,\%.$$

The turbine was tested at 6 different loads and the results which determined the efficiency curve of fig. 170 are given in table XLVIII. As to the accuracy of the method used to calculate the discharge, table XLIX shows comparative figures, determined at the tests of the Shawinigan Water & Power Company's 6000 HP turbines.

Table XLII.

Distance from Centre (Feet)	No. of Division	Distance from Circumference (Feet)	Surface of Cercle (sq. ft.)
2.890	1	0.110	26.23
2.650	2	0.350	22.06
2.400	3	0.600	18.10
2.075	4	0.925	13.53
1.737	5	1.263	9.48
1.378	6	1.622	5.96
0.574	7	2.453	1,04
0	8	3.000	0
0.574	9	3.574	1.04
1.378	10	4.378	5.96
1.737	11	4.737	9.48
2.075	12	5.075	13.53
2.400	13	5.400	18.10
2.650	14	5.650	22.06
2.890	15	5.890	26.23

Table XLIII. — *Gauging of pitot tube (vertical traverse).*

No. of Tube	Height of Water Level	Mean Correction
1	0.0	
2	0.0	
3	— 0.3	
4	— 0.5	— 0.19
5	— 0.3	
6	— 0.2	
7	0.0	
8	— 0.2	

Table XLIV. — *Gauging of pitot tube (horizontal traverse).*

No. of Tube	Height of Water Level	Mean Correction
8	0.0	
7	+ 0.1	
6	0.0	
5	— 0.6	— 0.10
4	— 0.5	
3	— 0.3	
2	+ 0.2	
1	+ 0.3	

Table XLV. — *Results of tests. Vertical traverse.*

No. of Division	1st test		2d test		3rd test		Mean Pitot	Mean Static	Difference P—S	Corrected (+ 0.19)	Head (feet)	Velocity (feet/sec)
	Level Pitot Tube	Level Static Tube	Level Pitot Tube	Level Static Tube	Level Pitot Tube	Level Static Tube						
	inches	inches	inches	inches	inches	inches	inches	inches	inches	inches		
1	50.7	39.5	49.7	40.0	50.3	39.9	50.23	39.80	10.43	10.62	0.885	7.55
2	50.0	39.8	52.0	39.7	54.5	39.6	52.16	39.70	12.46	12.65	1.056	8.22
3	54.1	39.5	55.9	39.1	58.6	38.9	56.20	39.16	17.04	17.23	1.436	9.62
4	58.2	39.1	56.9	39.3	57.5	39.5	57.53	39.30	18.23	18.42	1.535	9.95
5	59.4	38.8	59.3	38.7	60.8	38.8	59.83	38.76	21.07	21.16	1.771	10.68
6	61.4	38.5	54.2	31.5	54.7	31.3	56.76	33.76	23.00	23.19	1.932	11.16
7	52.6	31.1	52.6	30.8	53.9	31.2	53.03	31.03	22.00	22.19	1.849	10.90
8	53.6	31.6	52.9	31.3	53.7	31.2	53.40	31.36	22.04	22.23	1.852	10.91
9	52.2	31.2	53.1	31.3	53.3	31.5	52.86	31.33	21.53	21.72	1.811	10.82
10	55.0	31.4	53.8	31.3	51.5	31.5	53.43	31.40	22.03	22.22	1.852	10.91
11	51.6	31.3	54.2	30.9	51.1	31.3	52.30	31.16	21.14	21.33	1.777	10.71
12	51.9	31.3	49.1	31.6	48.1	31.7	49.70	31.53	18.17	18.36	1.530	9.93
13	50.7	31.5	50.3	31.4	46.8	31.8	49.26	31.56	17.70	17.89	1.491	9.80
14	48.0	31.4	47.6	31.6	46.4	31.9	47.33	31.63	15.70	15.89	1.324	9.23
15	44.4	31.7	43.1	31.8	44.0	31.7	43.83	31.73	12.10	12.29	1.023	8.11

Table LXVI. — *Results of tests. Horizontal traverse.*

No. of Division	1st test		2d test		3rd test		Mean Pitot	Mean Static	Difference P—S	Corrected (+ 0.10)	Head (feet)	Velocity (feet/sec)
	Level Pitot Tube	Level Static Tube	Level Pitot Tube	Level Static Tube	Level Pitot Tube	Level Static Tube						
	inches	inches	inches	inches	inches	inches	inches	inches	inches	inches		
1	48.1	35.8	47.6	35.7	45.1	35.9	46.93	35.80	11.13	11.23	0.936	7.76
2	49.5	35.3	47.6	35.3	48.8	35.9	48.63	35.50	13.13	13.23	1.103	8.41
3	51.6	35.0	48.5	35.9	52.0	34.5	50.70	35.13	15.57	15.67	1.306	9.17
4	51.7	35.0	53.7	34.7	49.9	35.6	51.76	35.10	16.66	16.76	1.398	9.48
5	54.4	34.7	52.9	34.7	55.3	34.5	54.20	34.63	19.57	19.67	1.639	10.28
6	57.1	34.5	54.5	35.1	54.4	34.9	55.33	34.83	10.50	20.60	1.717	10.54
7	56.8	34.5	55.8	34.6	55.5	34.4	56.03	34.50	21.53	21.63	1.803	10.80
8	56.3	34.4	55.8	34.4	57.9	34.3	56.66	34.36	22.30	22.40	1.866	10.96
9	59.3	34.5	57.3	34.1	56.3	34.2	57.63	34.36	23.37	23.47	1.955	11.22
10	57.8	34.2	55.9	34.3	58.6	34.0	57.43	34.16	23.27	23.37	1.947	11.19
11	56.2	34.3	55.7	34.6	56.4	34.3	56.10	34.40	21.70	21.80	1.817	10.84
12	55.0	34.8	53.9	34.5	55.2	34.5	54.70	34.60	20.10	20.20	1.684	10.44
13	51.8	35.0	52.7	34.9	51.9	34.9	52.13	34.93	17.20	17.30	1.442	9.64
14	49.1	35.1	49.5	34.8	48.9	35.1	49.16	35.00	14.16	14.26	1.189	8.75
15	43.6	35.4	44.1	35.0	44.0	35.6	43.90	35.33	8.57	8.67	0.723	6.83

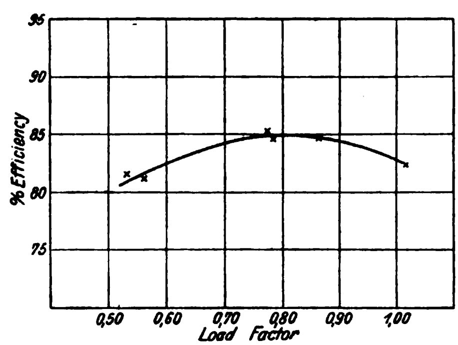


Fig. 170. Turbine Efficiency Curve.

Table XLVII. — *Switchboard readings during tests.*

(Generator 3,000 kw.; 16 Poles, 300 R. P. M.)

Frequency	Load factor	Amperes	Amperes	Amperes	Volts	Volts	Watts	Watts	KW at Switchboard	Generator efficiency	HP Turbine
40	100	2.3	2.33	2.25	109	109	218	216			
40	100	2.31	2.33	2.28	110.5	110.3	223	217			
40	100	2.31	2.32	2.3	110.1	110.1	224	218	2.783.11	96.75	3.851
40	100	2.35	2.33	2.25	110.0	110.0	223	218			
40	100	2.30	2.34	2.24	110.5	110.5	220	214			
40	100	2.35	2.32	2.23	110.2	110.2	225	213			
40	99	2.35	2.33	2.22	111.0	111.0	230	215			
40	99	2.32	2.36	2.28	110.8	110.8	218	220	2.803.72	96.75	3.882
40	100	2.33	2.34	2.28	110.7	110.7	225	218			
40	100	2.30	2.32	2.26	110.8	110.8	223	217			

Table XLVIII. — *Determination of turbine efficiency at six different loads.*

Tests	KW at Switch-board	Efficiency of Generator	HP of Turbine	Mean HP of Turbine	Load Factor	Turbine Efficiency
1 a	2002	95.5	2810	2817.8	$\frac{2817.8}{5000} = 56.35\%$	81.04%
1 b	2013	95.5	2825.5			
2 a	2817.6	96.75	3903.8	3908	78.16%	84.61
2 b	2823.7	96.75	3912			
3 a	3112	97.1	4296	4307	86.14	84.63
3 b	3128.4	97.1	4318			
4 a	2783.1	96.75	3851	3866.5	77.33	85.26
4 b	2803.7	96.75	3882			
5 a	3679	97.25	5071	5075.8	101.52	82.38
5 b	3686	97.25	5080.7			
6 a	1854	95.3	2607	2646.5	52.93	81.57
6 b	1910	95.3	2686			

Table XLIX.

Curve	Results by indicated Method	Results by Formulae of Discharge over Weirs		
		Francis	Bazin	Fteley & Stearns
1st test	204.7	200.3	200.3	201.4
Mean of 3 other tests	263.6	257.3	261.3	262.3

105. Discharge Measurements by Means of Venturi Meter.

Venturi meters may be used to measure the quantity of water supplied to a turbine by an arrangement as shown in fig 171.

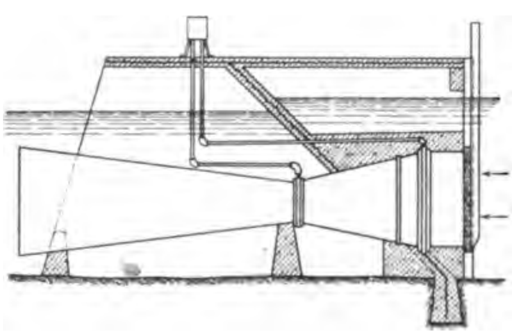


Fig. 171. Venturi Meter used as a Head Gate.

The principle of the Venturi meter is as follows: Suppose that water flows from a tank through a well rounded outlet into a pipe as shown in fig 172, the head required to give the velocity to the water V_a is

$$H_a = \frac{V_a^2}{2g}.$$

The head lost at the throat will be determined by the relation

$$H_b = \frac{V_b^2}{2g}.$$

The Venturi head or H_v will be determined by the relation

$$H_v = \frac{V_b^2 - V_a^2}{2g} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (1)$$

If R = ratio of cross-sectional areas A and B, then

$$V_a = \frac{V_b}{R}.$$

Therefore, (1) may also be written

$$H_v = \frac{V_b^2 - \frac{V_b^2}{R^2}}{2g} = \frac{V_b^2\left(1 - \frac{1}{R^2}\right)}{2g}$$

from which the value of V_b in function of H_v is found:

$$V_b = \sqrt{\frac{2gH_v}{1 - \frac{1}{R^2}}} = \sqrt{\frac{R^2}{R^2 - 1}}\,\sqrt{2gH_v} \quad . \quad . \quad . \quad . \quad (2)$$

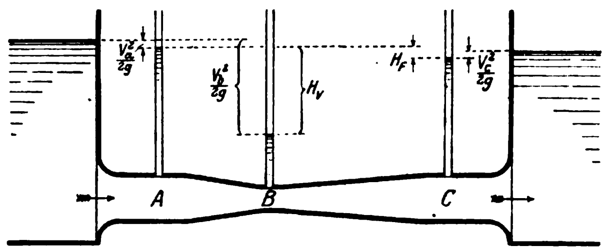


Fig. 172. The Venturi Meter.

It is observed, then, that (2) gives a direct relation between throat velocity and Venturi head providing there are no losses due to friction or eddy currents. In fig 172, H_f represents the friction head. Experience shows that the Venturi meter is accurate to within, 2 % approx.

106. Measurement of Water supplied to Impulse Wheels.

There are very few records of water quantities used by impulse wheels, and therefore the efficiency of such a machine is a matter of difficult computation. Prof. W. Eckart has made some very original investigations in this line, and the mention of the method developed by him in his experiments is of unusual interest. These experiments were made at Deer Creek. Mr. Eckart developed a special pitot tube designed to measure velocity heads in impulse wheel nozzles, and the auxiliary apparatus consisted in a contraction gauge arranged to measure the diameter of the jet (fig. 173) and Crosby pressure gauges in the main pipe just back of the nozzle. Prof. Eckart describes the method as follows: "The impulse wheel of the Deer Creek plant is regulated by a deflecting nozzle, which is provided with a needle for shutting down the flow by hand. Pitot tube tests were made for four positions of the needle in the nozzle, the jet velocity for each position being calculated by the formula:

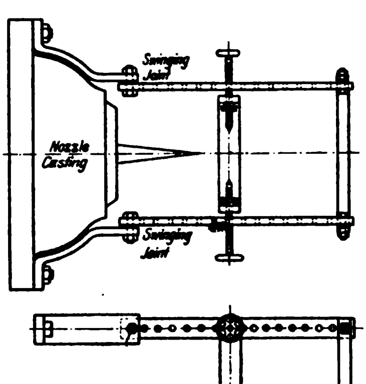


Fig. 173.

$$V = \sqrt{2gh}$$

where V = velocity in feet per second, and
h = head in feet read from the Pitot tube.

The values of h at various distances from the center of the jet, at its minimum cross-section, were reduced to a mean value by the following calculation, which also gives the total discharge. The velocity determined by the pitot tube was assumed to be uniform in all points equidistant from the center of the jet, so that at any distance r from the center, we have an elementary annular ring of width dr, length $2\pi r$, and area $2\pi r dr$, throughout which the velocity V is uniform. The volume of discharge for this ring would be $2\pi r dr V$ or dQ whence:

$$Q = 2\pi \int_0^R V r dr = va$$

where $Q =$ discharge in cubic feet per second,
$v =$ mean velocity of jet in feet per second, and
$a =$ area of the jet in square feet.

Hence:

$$v = 2\pi \int_0^R \frac{V r dr}{a}.$$

Plotting a curve having for its abscissae the values of r, and for its ordinates the corresponding values of $2\pi r V$, the area under this curve will be equal to the total volume of discharge of the jet or:

$$2\pi \int_0^R V r dr$$

from which the mean velocity may be obtained by dividing by the area of the jet. This curve of values for $2\pi r V$ may be obtained graphically from the curve of velocities as follows: Draw the velocity curve with ordinates equal to the velocities determined by the pitot tube, and abscissas equal to the distance from the jet at which measurements were taken, fig. 174. The curve may be constructed upon the same base, and the same scale for ordinates may be used if the value of $2\pi r$ at the outer edge of the jet be taken as unit, so that:

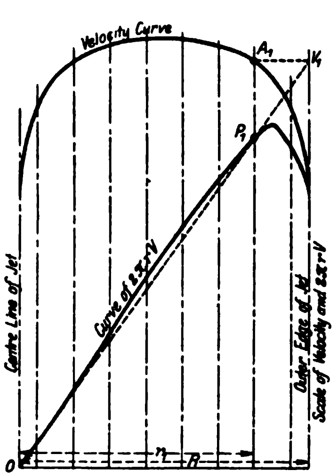


Fig. 174.

$$2\pi r V = V_1$$

then the length of the ordinate for the curve OP or for any point A_1 on the velocity curve where the velocity equals V_1 would be found graphically by the intersection of the diagonal OV and the ordinate through A_1. Other points on the curve being located in the same manner, the area beneath it may be determined by planimeter, and the mean velocity by dividing this area by area of the jet. The total power of the jet will be the sum of the kinetic energies per second of all elementary portions of the cross section. If E is the power in foot pounds per second, and e the power in foot pounds per second per

unit of area, then the total power for any elementary annular ring would be

$$dE = 2\pi r e dr$$

and for the whole jet

$$E = 2\pi \int_0^R e r dr = e_i a$$

where e_i = mean power of the jet in feet pounds per second per unit of area, and

a = total area of the jet in square feet.

This may be treated graphically by plotting the energy curve in place of the velocity curve.

107. Bearings.

There are two classes of bearings: bearings for horizontal shafts and bearings for vertical shafts. As both types are different, each will be considered separately. A horizontal bearing consists of a collar on the revolving shaft bearing against a fixed surface. In practice, several forms of bearings are used, but it should be noted that whenever possible, the bearings are preferably not submerged, unless they are of the lignum vitae type. These are sometimes lubricated by water only, whereas, in other cases, lubrication is effected with forced grease; for this purpose the boxes are water tight, and flow into them is prevented by stuffing boxes located at each end of the main box. The bearings must be always accessible, either by man holes, or by other means. When the bearing is not submerged, and in order to prevent as far as possible overheating of the oil, the oil chamber about the bearing should be as large as practically possible, otherwise special cooling devices must be installed. Experience shows that the bearing secured to the wheel case gives more satisfactory results than one having a special support, the latter being liable to get out of alignment. As to the lining of cast iron bearing boxes, white metal bushing give better results than the ordinary bronze bushing. The object of the thrust bearing is to counterbalance the thrust caused by the action of the turbine when the discharge is in one direction. Mr. A. Giesler[1] describes as follows the thrust bearing device used in connection with the Snoqualmie Falls turbine.

"Single wheel horizontal shaft units are relatively infrequent in turbine practice, especially in large sizes, where the thrust of a single runner is large enough to require careful consideration. The thrust is made of two parts: first, that due to the static pressure or effective head of water at the various points of the runner surface; and second, that due to the deflection of the water from a purely radial path through the wheel. As concerns the first part, the front face of the wheel is pressed upon by a pressure varying from the supply head at the outer circumference to the discharge pressure (vacuum) at the inner edge of the vanes, which latter extends over the whole central area of the runner (and shaft extension). The rear face of the runner is subjected to the pressure of water leaking

[1] Engineering News — March 29 — 1906.

through the radial air-gap between casing and runner, substantially equal to the supply head. This greatly overbalances the pressure on the front face, and the resultant thrust is to the right in fig. 175 (toward the draft tube). The discharge ends of the vanes, being curved transversely, also have a pressure component directed toward the right. The velocity effect produces a thrust directed toward the left, but this is very small and does not materially reduce the pressure thrust. By far the larger part of the pressure thrust is eliminated by venting the space back of the runner into the discharge space. Six holes through the wheel near the shaft, have this function. The water leaking in through the air-gap is continuously discharged through these vents into the draft tube, and the accumulation of any large static pressure back of the wheel is thereby avoided. The average pressure on the front of the runner however, is always lower, and the resultant thrust is therefore toward the draft-tube, though its amount varies considerably, being greatest for full gate opening. This thrust is taken up by the balancing piston immediately back of the rear head of the wheel case, and the ultimate balance and adjustment of position is accomplished by the collar thrust bearing behind the balancing piston. The balancing piston is a forged enlargement of the shaft, finished to a diameter of 17 inches, which works in a brass sleeve set in a hublike projection on the back of the wheel-housing.

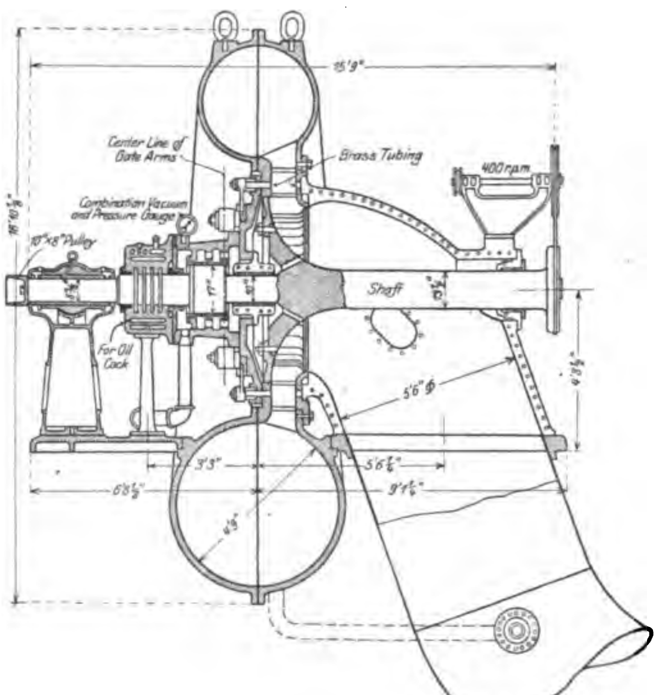
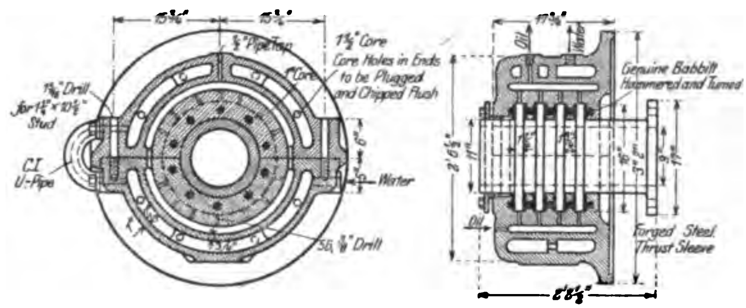


Fig. 175. Section of Snoqualmie Falls Reaction Turbine.

The inside of the sleeve has six circumferential grooves, each one inch wide and one quarter inch deep, as water packing. The chamber in front of the piston communicates by a pipe (containing a strainer) with the supply casing of the water wheel, and therefore receives the full pressure of the supply head. The chamber back of the piston is drained to the draft tube, so as to carry off any leakage past the piston. The device thus produces a constant thrust on the piston, directed toward the left. By throttling the pressure pipe, this thrust can be adjusted as desired. The thrust bearing shown in fig. 175 and in detail in fig. 176, consists of a group of four collars on the shaft, working in a babbited thrust block which is bolted to the back of the wheel housing. The collars are formed on a steel sleeve which fits over the shaft and is bolted to the rear face of the balancing piston; this

makes it possible, when collars are worn out, to renew the bearing by dismounting the thrust block and placing a new sleeve. The thrust bearing is lubricated by oil immersion. An oil chamber is cored in the block,



Cross Section. Longitudinal Section.

Fig. 176. Thrust-Bearing-Snoqualmie Wheels.

and communicates by numerous oil holes with the bearing faces; a constant flow of oil is maintained by means of oil supply and drain pipes. Concentric with the oil chamber and outside of it, a water chamber is cored in the block. Cooling water is supplied to this chamber by a pipe from the pressure side of the turbine, and drains from the top of the bearing through a drain pipe to the draft tube. A U-pipe attached at one side of the bearing forms connection between the water chambers of the upper and lower halves of the block. This detail avoids making the connection by a hole through the joint face, which would allow leakage of water into the oil space and into the bearing. The balancing piston is so proportioned and the pressure supply pipe is throttled to such a point as to give exact balance (i. e. with zero thrust in the thrust bearing) at about half to five eights the full output of the wheel. At larger power there will be an unbalanced thrust to the right, and at a smaller output

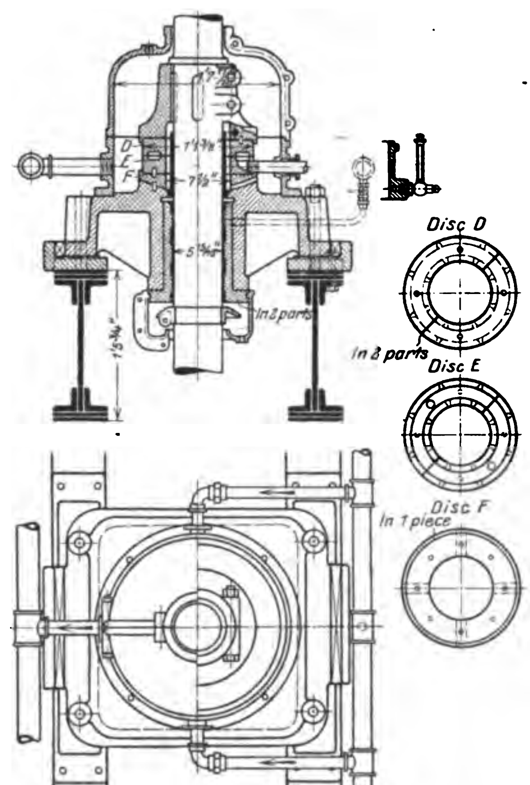


Fig. 177. Oil-Pressure Step-Bearing for Vertical Shaft.

to the left, which are taken by the thrust bearings. The maximum thrust on the collars is about 25 000 lbs. The collars are $2\frac{1}{2}$ inches high ($2\frac{3}{8}$ inches effective) by $13\frac{1}{2}$ inches mean diameter, giving a total effective bearing

area on 4 collars of 418 square inches. The maximum collar pressure is thus about 60 lbs. per sq. in."

If twin turbines are used on a single shaft, it is best to arrange their discharge in opposite directions, the thrust of one being then counterbalanced by the thrust of the other. Vertical pressure or step bearings for vertical shaft turbines are generally supported above water, where they are readily accessible. Fig. 177 shows a bearing designed by M. V. Gelpke. The oil under pressure is acting between the discs, and the result of its upward force is the counterbalancing of the downward forces, and oil grooves are provided so that it will operate as an ordinary step bearing. The thrust bearing used at the Niagara Falls Power Company's plant is illustrated in fig. 178. The discs were made of close grained charcoal iron of 25 000 lbs. tensile strength, and the mating faces were scraped to a

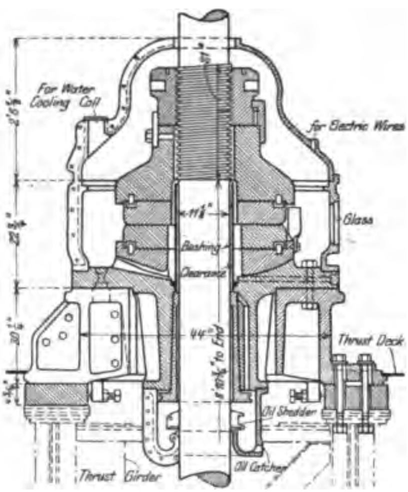


Fig. 178. Thrust or Hanging Bearing of the Wheels of the Niagara Falls Power Co.

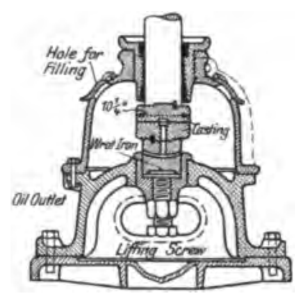


Fig. 179. Pivot.

bearing. Three different types of thrust bearings were constructed, their difference consisting in the method of lubrication employed: forced lubrication, a combination of forced and self-lubrication, and self-lubrication alone. In the combination bearing, arrangements are made to utilize two different pressures: the low pressure oil emanating from a general lubricating system, and the high pressure oil from a special pump introducing the oil between the discs under 400 lbs. pressure. The thrust discs are enclosed in a casing which is provided with two slight holes, in diametrically opposite positions, permitting the observation of the condition of the bearing and the temperature of the oil, by means of a thermometer hanging in the oil. The oil is cooled by water circulating pipes inside the casing. Guide bearings are always used if the shaft is somewhat long, and when a thrust bearing such as those described is found to be too expensive, a pivot, as illustrated in fig. 179 may be used at the lower end of the vertical shaft, thereby supporting the whole turbine shaft. The stationary disc is fastened by means of steel dowel pins to a third disc which has a spherical seat scraped to fit a support. This spherical disc allows slight deviations from the vertical.

108. Fly-Wheels.

To insure uniform operation of alternators working in parallel, as well as to take care of the fluctuations of load and fluctuations in water pressure, it is necessary to apply a fly-wheel which is independent of the centrifugal regulator. In large units, the inertia of the rotating parts may be sufficient, but in many cases, a fly-wheel is required. The allowable coefficient of irregularity in alternators running in parallel is determined by the expression:

$$\frac{V_{max} - V_{min}}{2 V_{av}}$$

the value of which for best results, was generally taken as $\frac{1}{200}$; so it can be admitted that

$$\frac{V_{max} - V_{min}}{2 V_{av}} = 0.005.$$

Calling

E = energy stored in fly-wheel, in ft. lbs.,
W = weight of wheel in lbs.,
V = velocity or speed in feet per sec., and
g = constant of gravity = 32.16;

the stored energy in the fly-wheel is determined by the expression:

$$E = \frac{W}{g}\left(\frac{V^2_{max} - V^2_{min}}{2}\right) \text{ ft. lbs.} \quad \dots\dots\dots \quad (1)$$

The peripheral speed of the wheel in function of the radius R is

$$V = 2\pi R V_{sec} \text{ ft. per sec.}$$

in which V_{sec} is the speed in revolutions per second.

Substituting in (1)

$$E = \frac{W}{32.16}\left[\frac{4\pi^2 R^2 (V^2_{sec\,max} - V^2_{sec\,min})}{2}\right] \text{ ft. lbs.} \quad \dots\dots \quad (2)$$

One horse power is equal to 550 ft. lbs. per sec., and may be written in function of kilowatt-sec. as follows:

$$\frac{1 \text{ Kw. sec}}{0.746 \text{ Kw. sec.}} = \frac{x}{550 \text{ ft. lbs. sec.}} \quad \text{or} \quad x \approx 740 \text{ ft. lbs.};$$

establishing the approximate equality:

$$E = m \text{ Kw. sec.} = 740 \text{ ft. lbs.} \times m; \quad \text{i. e.: } \frac{E}{m} = 1 \text{ Kw. sec.}$$

Formula (2) may be written:

$$W R^2 = \frac{16 E}{\pi^2 (V^2_{sec\,max} - V^2_{sec\,min})} \quad \dots\dots\dots \quad (3)$$

The safe peripheral velocities of fly-wheels of different materials are:

Cast iron = 30 to 40 ft. per sec.,
Steel = 120 to 150 ft. per sec.,
Solid steel = 300 ft. per sec. maximum.

The length of the penstock affects the revolving parts of a turbine, so that the WR^2 effect may be written as follows in English measure:

$$WR^2 = 16\,232 \frac{P \times L(v_1 - v_2)}{ps^2 \delta} \quad \ldots \ldots \ldots \quad (4)$$

and the energy absorbed or given out by the revolving parts due to a change in speed may be written:

$$E = \pm \frac{WR^2 s^2 \delta}{2936.4} \quad \ldots \ldots \ldots \ldots \quad (5)$$

where the negative sign indicates that the revolving parts absorb energy upon a rise in speed. The time required to change the gate-opening is found by:

$$T = 0.0201 \frac{L(v_1 - v_2)}{p} \quad \ldots \ldots \ldots \ldots \quad (6)$$

in the above (see page 93):

W = combined weights of the revolving parts of the turbine,
R = radius of gyration of revolving parts,
s = speed of turbine in r. p. m.,
L = length of penstock in feet,
P = horse power of turbine,
$p = 0.0201 \frac{L(v_1 - v_2)}{T}$ = rise in pressure due to closing of gate,
v_1 = normal velocity of water in penstock,
v_2 = velocity after change in gate opening, and
δ = percentage of speed variation.

Let it be required to calculate the WR^2 of the revolving parts of a 5000 HP turbine, the penstock being 1000 ft. long, with a normal velocity of flow in the same of 6 ft. per sec. The allowable speed variation from full to no load is limited to 8% and the rise in pressure in the penstock should not exceed 60 lbs. per square inch when full load is thrown off. The speed is 250 r. p. m. By substitution in (4), and considering that $v_2 = 0$

$$WR^2 = 16\,232 \frac{5000 \times 1000 \times 6}{60 \times 250^2 \times 0.08} = 1\,623\,200.$$

The minimum time required to close the gate in order that the pressure of 60 lbs. per sq. in. be not exceeded is:

$$T = 0.0201 \frac{1000 \times 6}{60} = 2.01 \text{ sec.}$$

109. Couplings.

Hydraulic and electric machines are generally coupled together by some mechanical connection, known respectively as rigid, flexible and clutch couplings, used for both horizontal and vertical machines. The

flexible or elastic coupling is the one that is generally used for hydro-electric work, as it tends to reduce shocks due to fluctuations in the angular speed of the machines, besides allowing for differences in alignment. There are many forms of flexible couplings, all of which present more or less simplicity of design. There is a coupling consisting of a wheel and a spider, one rigidly mounted on each shaft. The pins, with which both are provided, are connected by springs. A coupling which is strong, flexible and very simple is the Zodel joint. It consists of two flanged plates, located one inside of the other, and both provided with slots, through which a cotton or leather belt is wound in and out. The plates have the appearance of being sewed together. Friction clutch couplings have the advantage over any other clutch coupling of permitting the throwing on of a load shaft without stopping the driver, and also of being readily disconnected, so that generators can be stopped without interfering with the turbines. The principal advantages lies in the fact that the load of machine can be put on slowly and without shock. In small plants the inertia of a coupling is sufficient to act as a fly wheel, thereby counterbalancing the fluctuations of the load.

110. Relief Valves and By-Passes.

These are used on long pipe lines in order to allow the escape of excess energy produced by the closing of turbine gates or other causes. The most efficient types in use are the Lombard hydraulic relief valve (diagramatically represented in fig. 180) and the Sturgess automatic relief valve (illustrated in fig. 181).

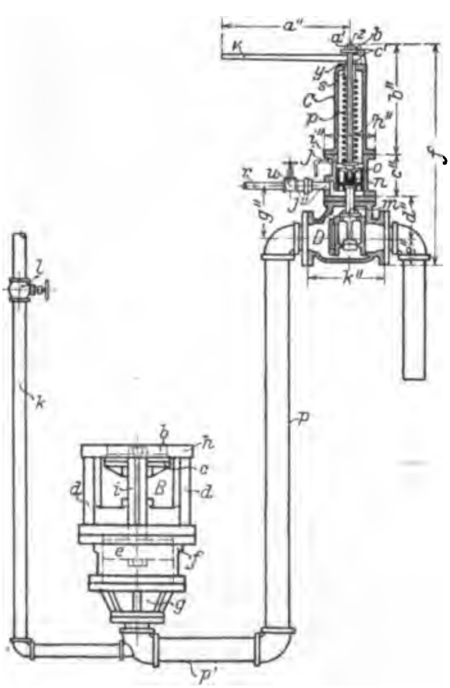


Fig. 180. Lombard Hydraulic Relief Valve.

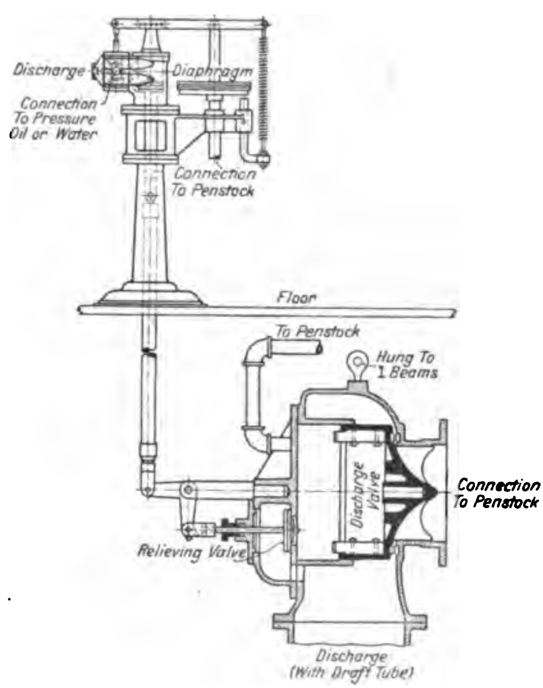


Fig. 181. Sturgess Relief Valve.

The Lombard valve consists of the following parts,[1] viz.: A valve disc *c* capable of motion to or from its seat, *b*, rigidly connected by means of a rod, *i*, with the piston, *f*, in the cylinder, *e*. The whole valve is bolted to a flange upon the supply pipe, *d*, wherein the pressure is to be controlled. The area of piston, *f*, is somewhat greater than that of the valve disc, *c*, so that when water at the same pressure is behind the piston and in front of the valve, there is a positive and strong tendency to hold the valve closed. For the purpose of allowing the valve disc, *c*, to open at proper times to relieve excess pressure in the supply pipe, *d*, there is provided a regulating waste valve, *C*. This valve is opened or closed by a piston, *n*, opposed by an oblong and strong spiral spring, *p*. Piston, *n*, is a loose fit in its cylinder, *o*, so that it moves upward freely in response to the least excess in pressure upward due to the water in the cylinder, *o*, opposed to the downward pressure of the spring, *p*. The piston, *n*, is connected by means of the stem, *m*, with a double seated balanced valve, *d*, which of course, opens simultaneously with any upward movement of the piston. Water under existing pressure is admitted into the cylinder, *e*, through the pipe, *k*, and throttle valve, *l*.

The essential element in the Sturgess automatic relief valve is a large, very sensitive diaphragm of special construction. This is under the influence of the water pressure in the pipe line and its movements are communicated to a small pilot valve controlling a hydraulic cylinder which in turn operates the relieving valve on the relief valve proper. After the pressure in the pipe line is restored to normal, the relief valve gradually closes automatically. The action of this valve is almost instantaneous, and it will fully open on a very small rise of pressure. These valves can either be made in self contained form, or the sensitive parts (diaphragm, pilot valve, and hydraulic cylinder) may be mounted on a pedestal placed in the power house, and the relief valve proper attached to the penstock or wheel casing, a rod or link being provided to connect the two (as in fig. 181). The by-pass consists generally of a valve the opening of which is sufficient to allow the escape of the total volume of water demanded for the operation of the turbine. Its operation is contrary in effect to the operation of the turbine gate, the opening area of the by-pass increasing in the same proportion as the turbine gate opening decreases. In this way, shocks due to water hammer will never occur, and the by-pass is used to its best advantage in connection with impulse turbines. If economy in water consumption is imperative, a temporary by-pass may be used, which differs from the other in that it closes automatically and slowly as soon as the closing movement of the turbine gate ceases.

111. Speed Regulation.

In all commercial plants, where the load is variable, the prime factor to be considered is a uniform speed of the revolving machinery, a condition which requires automatic devices for controlling the admission of water to the wheels; this is accomplished by some form of governor which acts upon the turbine gates, closing or opening these, whenever the velo-

[1] Lombard Bulletin. No. 101.

city exceeds or falls below the normal. Regulation of turbines supplied by long penstocks is difficult, as a partial closure of the turbine gates results in a sudden increase of water pressure within the penstock, and the contrary operation produces a decrease in the penstock pressure and consequently a partial vacuum. It is also necessary to obviate these conditions by the use of pressure relief valves, by-passes, standpipes, air-chambers or fly wheels.

112. Governors.

The generator voltage being directly proportional to the speed, it is of prime importance to maintain constant speed in the turbine, a condition which is assured by the governor.

Governors are classified according to the type of their relay, sometimes called servo-motor, and which consists of an auxiliary motor or other device, using mechanical, hydraulic or electrical power, or oil pressure, and transmitting energy of the sensitive centrifugal governor, and controlled by it in the operation of the gates.

113. Governors. Mechanical Type.

The power for a mechanical governor is usually furnished by the turbine. A governor which is much in vogue is the "Repogle", described as follows by its designer: In the diagram, fig. 182 *A* is a spherical pulley with its shaft turned down and threaded as at *X*. *B* and *B* are oppositely revolving concave disks lined with leather. *C* and *C* are lignum-vitae pins flush with the leather. *D* and *D* are compression springs for causing the necessary pressure between the disks and the sphere. *E* and *E* are governor balls so poised as to require the weight of *A* to balance them at normal speed. *F* is a loose collar to allow independent revolution of the balls *E*, *E*. *G* is the point of connection between *A* and the gates or valves of the motor to be governed. *X* is the relay device, and is for the purpose of preventing racing, also for the purpose of properly dividing the load in parallel units. *Z* is a stationery spindle or connecting link between collar *F* and the threaded shaft or pulley *A*. *Z* is only stationery in reference to revolution, as it rises or falls with the variations of the governor balls. Fig. 183 shows a Woodward simple mechanical governor. Letters *a* and *b* show two friction pans which are loose on the shaft, the upper one being supported by a groove in the hub, and the lower one by an adjustable step-bearing. The friction wheel which is double faced and bevelled to fit the pans, is keyed to the shaft. This shaft and friction wheel run continuously and have a slight endwise movement. Being supported by lugs on the ball arm, they rise and fall

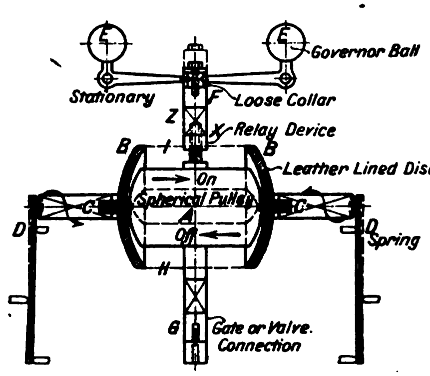


Fig. 182. Repogle Governor.

as the position of the balls varies with the speed. The power necessary to move both the centrifugal governor balls and the relay is transmitted through a belt to the pulley *P*. With a normal speed, the pans are stationary and the friction wheel revolves freely. With a change of speed the friction wheel is brought against the upper or lower pan as the speed is either slow or fast. The pan is thereby revolved, and being connected with the gates, operation result in the proper direction until the speed is normal again. As the gate opens, the nut *d* travels along the screw *e* which is driven through gearing by the main governor shaft, and as the gate reacts, the nut *d* coming in contact with the lever *f*, throws the vertical shaft upward, and puts the governor out of commission.

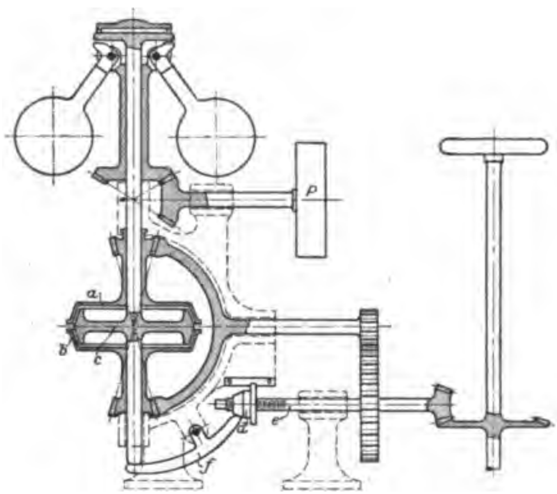


Fig. 183. Diagramatic Section of Woodward Simple Mechanical Governor.

If the friction load is comparatively small, the water wheel being direct connected to a single machine, the relative change in load, and consequently the possible change in speed, is much larger, in which case this type of governor is subject to racing of the wheel, a condition that can be avoided by the use of the Woodward compensating governor, shown in fig. 184. The pulley *P* operates the relay, while the centrifugal governor balls are moved by the pulley *q* which is independent belt-connected to the turbine shaft. The centrifugal governor balls are supported by a hallow shaft and two lugs placed on the ball arms connect with the spindle *f* which rises and falls.

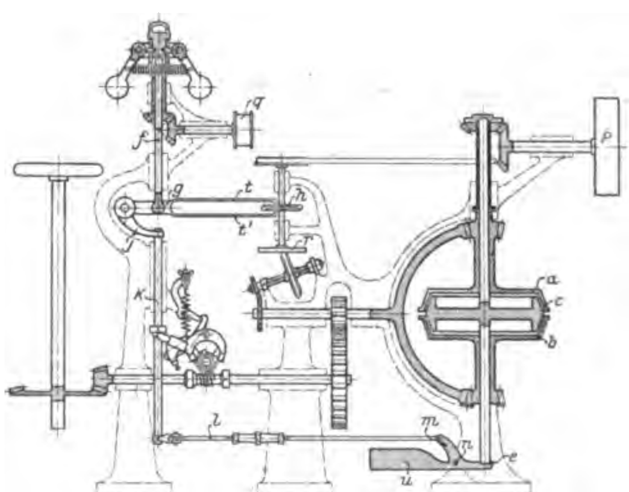


Fig. 184. Diagramatic Section of Woodward Vertical Compensating Mechanical Governor.

The centrifugal governor balls cause the spindle to rise or fall thereby changing the position of the tappet arm *g* and causes either of the tappets *tt*[1] to engage a double faced cam *h*. This cam is in continuous rotation as shown in the figure. The tappets are connected to a common suspension arm to which the

vertical spindle is attached. By means of the lever connections J and K, the tension rod l is connected to an eccentric at the bottom. In turn, the tension rod l is connected by the lever m with the vertical bearing e connected to the main shaft; u is a counterweight. The rotating cam engaging either tappet, the resulting movement turns the rocker shaft K, thereby raising or lowering which in turn engages the friction wheel against either pan as in the simple governor.

On the lower end of the cam shaft is a friction disk, r, which rests on a rawhide friction wheel on a diagonal shaft. The hub of the friction is threaded and fits loosely on the diagonal shaft which is normally at rest. The effect of the continually rotating friction disc upon the raw-hide wheel is evidently to cause it to travel along the threaded diagonal shaft to the center of the disc. When the governor moves to open or close the gate, the diagonal shaft which is geared to it, is turned, and the friction wheel is caused to travel along the shaft away from the center of the disc, and thus raise or lower the cam shaft so as to separate the cam from the tappet which is in action, before the gate has moved to far, thus preventing racing. As soon as the gate movement ceases the disc causes the friction wheel to return to the center of the disc along the threaded shaft. To prevent the governor from straining when the gate is fully open or closed, suitable cams are mounted on the stop shaft. When the gates are completely opened the cam engages the speed lever and holds it down so that it cannot raise the lower tappet sufficiently to engage the revolving cam; this does not, however, interfere with the upper tappet to prevent the closing of the gates, should the conditions demand. The closed gate stop acts in a similar manner on the upper tappet, but does not interfere with the lower tappet being engaged, should the conditions demand that the gate be opened. In addition to these stops, the governor is provided with a safety stop the function of which is to immediately close the gates should the speed governor stop through breakage of the belt or from any other cause.

114. Governors. Hydraulic Type.

These are operated by water or oil pressure and by means of a piston and a cylinder. The governor consists of the ordinary type of centrifugal balls put in motion by the turbine shaft, and connected to the gates. When oil is used, the necessary pressure is furnished by an oil force pump. The use of oil reduces to a minimum the wear of the interior working parts, which therefore do not require any separate lubrication. Governors of this kind are the Lombard, Sturgess and Glocker-White. The latter, fig. 185 is manufactured by the I. P. Morris Company, and the essential feature are the governor's centrifugal balls A, partially filled with mercury, and divided into two chambers when running at normal speed. When the speed increases, the centrifugal force causes the mercury to flow from the lower to the upper chamber; thus the center of gavity increases in a greater ratio than the speed. The action is transmitted through a system of levers to a small pilot valve, controlling a relay valve admitting oil to the cylinder, which in turn causes movement in the turbine gates. Fig. 186 shows a simple and efficient hydraulic governor

applied to an impulse wheel. Mr. Thurso describes it as follows: In the valve chamber *g* is the admission valve *r*, which is in the shape of a differential plunger, and is kept floating by the difference of water pressure existing in the spaces *a* and *b*. The pressure water after passing through the filter enters the valve chamber at *a*, and flows through the bore in the admission valve into the space *b*. The pressure in space *b* is regulated by the size of the vent hole opening, which is controlled by the regulating valve *v*, which in turn is moved by the centrifugal governor. At normal speed, the admission valve *r* is in the middle position as shown, the port *d* of the relay cylinder being closed. Water flows continuously from *a* through the bore in the admission valve *r* to the space *b* and from there through the vent valve to the space *c* and then escapes through the waste pipe shown. When the speed of the turbine decreases, the centrifugal governor raises the regulating valve *v*, the pressure in the space *b* is reduced, and the admission-valve raised, permitting the pressure

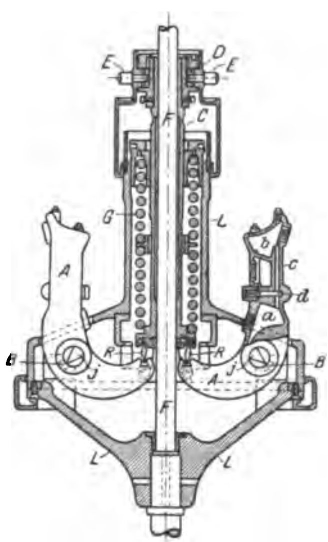


Fig. 185. Glocker-White Governor.

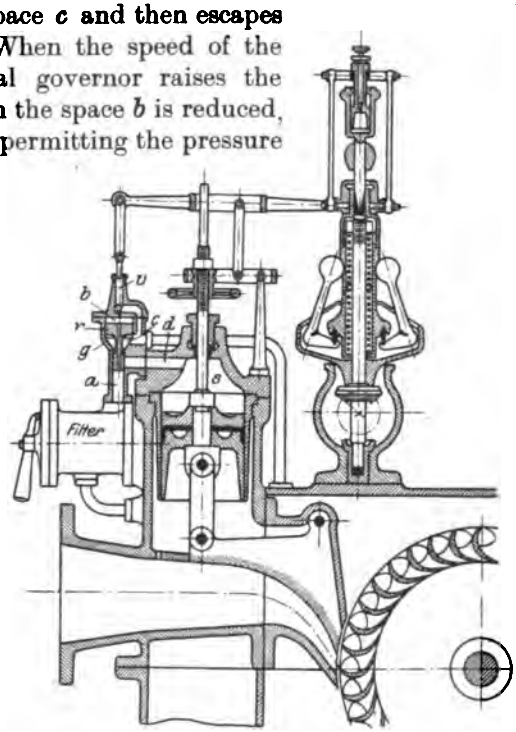


Fig. 186. Hydraulic Governor for Impulse Turbines. Built by Escher Wyss & Co., Zurich, Switzerland.

water to flow from *a* through the port *d* into the relay cylinder, forcing down the relay-piston and thus opening the tongue of the turbine nozzle, that is, increasing the gate opening. However as the fulcrum of the lever which moves the regulating valve *v*, is connected to the piston rod of the relay-piston, the downward movement of this piston returns or lowers the regulating valve *v*, which decreases the vent-hole opening, and thus causes an increase in the pressure in the space *b*, which forces the admission valve back to its middle position, and thus stops the motion of the relay piston and the tongue. With an increase in the speed of the turbine, the opposite action takes place, the regulating valve *v* being lowered and the pressure in the space *b* is increased. This forces the admission valve *r* down and permits the pressure water in the relay cylinder to escape through port *d*, space *c*, and the waste pipe shown; and the water pressure

existing in the space above the tongue and below the relay piston forces the latter upwards. The governor is very sensitive and quickacting on account of the small mass and inertia of the regulating valve *v*, the comparatively large areas of *a*, *b* and *c*, the floating state of the admission valve *r*, and the continuous flow of water through the valve chamber *g*.

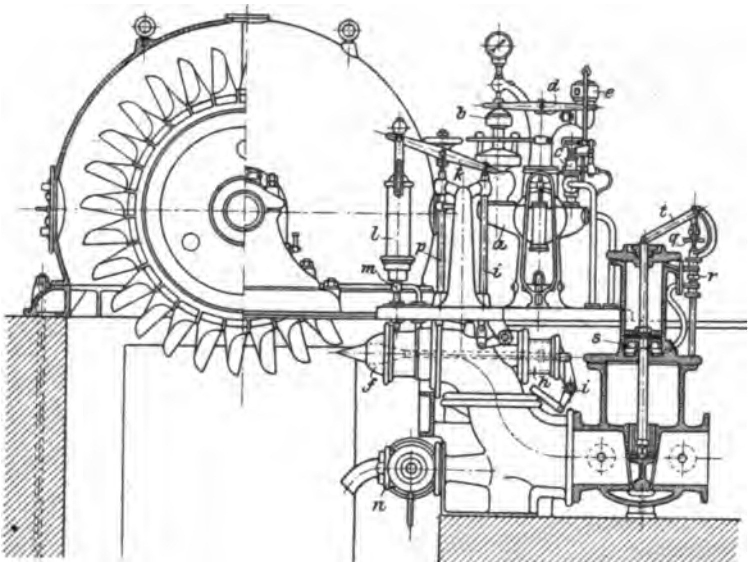


Fig. 187. Pelton Wheel and Details of Regulator.

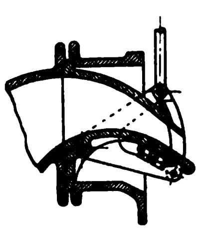


Fig. 188. Circular Nozzle closed by sliding Cut-Off.

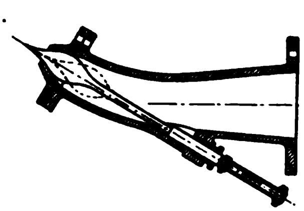


Fig. 189. Circular Nozzle with Long Needle passing through Sheath.

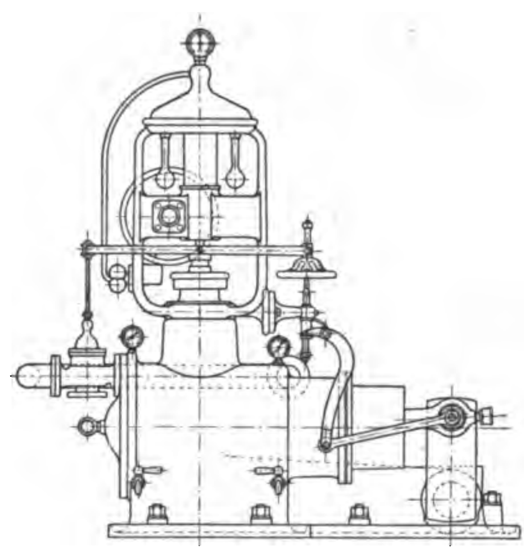


Fig. 190. Cachin Regulator.

In most cases the nozzle is stationary while the needle, which cuts off the water supply when moved forward is put in motion by the governor. This is illustrated in fig. 187. Fig. 188 shows a circular nozzle closed by a sliding cut-off, whereas fig. 189 shows a circular nozzle with needle passing through sheath.

Fig. 190 illustrated the Cachin type of hydraulic governor.

115. Governors. Electrical Type.

Turbines may also be controlled electrically from the switchboard, and the necessary power to put the gates in motion may be furnished by the generator coupled to the turbine as illustrated in fig. 191. *A* is the generator furnishing the current, *B* is a motor placed in or out of circuit by means of the mercury switch *D* put in motion by the governor balls *C*. The motor is made to run in the proper direction, or to reverse, according to the position of the switch.

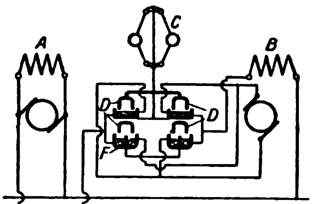


Fig. 191. Diagram of Electric Type Governor.

116. Regulating Gates.

The following gate arrangements have come into general use for controlling the admission of water into reaction turbines.

The *cylinder gate* consists of a smooth cylinder sliding closely between the distributor and the runner, and thereby regulating the flow of water into the buckets. The efficiency of turbines equipped with cylinder gates is high only when the gate is wide open; the „part gate" efficiency may be improved by the use of two or three story turbines which are regulated by a common cylinder gate. The details of a vertical turbine with cylinder gate is shown in fig. 192.

The *register gate* consists of a rotating cylinder having apertures which correspond with the outlet openings of the guide buckets. When the register gate is partly closed, eddy currents of considerable strength are produced, with a resultant decrease in part gate efficiency. It is sometimes placed inside or outside the guide-ring but in general, this form of gate has found little application of late.

The *wicket gate*. In this gate all the vanes move and thus increase or decrease the discharge openings of the buckets simultaneously. Probably this is the most satisfactory gate for medium or high heads. Fig. 142 illustrates a wicket gate of the improved New American turbine.

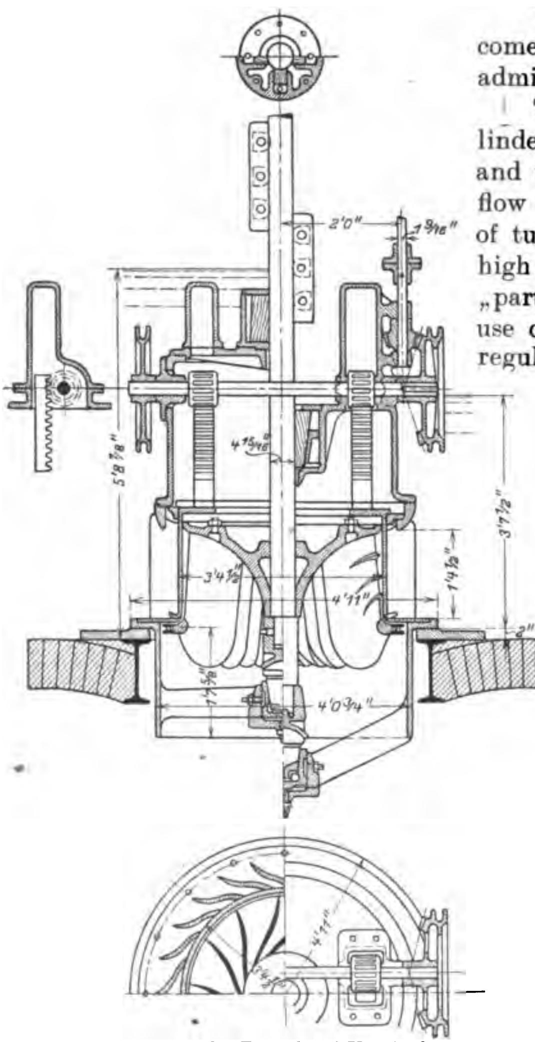


Fig. 192. Details of Vertical Turbine with Cylinder Gate.

Chapter VII. Power House and Substation Equipment.

117. Screens.

The design of screens does not in general receive a sufficient consideration, and it seems quite customary to neglect this important element of a power plant. The absence of a thorough knowledge of all its features may involve an unnecessary increase in operation cost.

The purpose of screens is to strain out of the water, such floating matter as leaves, bark, grass, wood debris, ice, etc., so as to prevent choking or damaging the turbines. As the speed of water in a head race is usually 2 to 3 feet per second, this speed must be gradually increased to the speed in the intake tube of the power house; it is obvious then, that the velocity of water through the screens must not be less than the velocity in the head race, i. e. 2 or 3 feet per second, and therefore the clear area of rack section for the passage of water must be calculated to permit the flow without loss of head.

Screens are made in such sections that can be easily handled, weighing not more than 500 pounds and from 4 to 6 ft. wide. These sections are made to slide down along supports or guides, and two adjacent sections are usually separated by *T* bars.

The principal object to be kept in view in the design of screens is their simplicity in construction. It must not be forgotten that they must often be taken out, dismounted, the bars straightened and put back into place.

Screens are divided into two classes; fine screens, composed of wrought iron or steel bars varying from $\frac{1}{4}''$ to $\frac{7}{16}''$ thickness, 3″ to 4″ width, and a clear spacing of $\frac{1}{2}''$ to 2″; coarse screens are made of bars $\frac{3}{4}''$ thick, 4″ to 5″ width and offering a clear opening of 3″. These bars are assembled in sections by means of rods, gas pipe separators and bolts, the simplest and most useful arrangements and details being shown in fig. 193.

To prevent their tipping over, they are fastened at the upper end by latches or clasps. It frequently happens that material will accumulate, completely stopping the flow if they are neglected for one reason or another; hence it is necessary, as a basis for resistance calculations, to consider that the entire screen for each bay acts as a dam.

In many instances, it will be found necessary to design supports, consisting either of a set of steel frames or simply of some *I* beams, conveniently spaced to resist the water pressure and anchored into the forebay piers. Such supports must be calculated so as to sustain the entire head of water.

To determine the clear screen area allowing the water flowing through them with a velocity of 2 or 3 feet per second, it can be assumed that

$25\,^0/_0$ of the total screen area is obstructed by the bars, rods, etc., so that the apparently required area, for a velocity v would be represented by

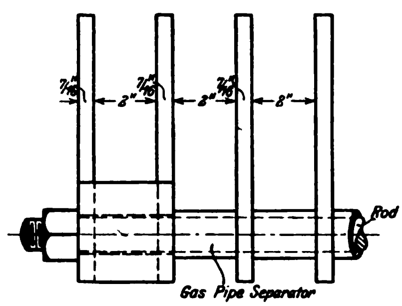
$$A = \frac{4}{3} \cdot \frac{Q}{v},$$

but as a contraction of water is produced, a correction must still be made to allow for diminished section of screen area bringing the actual screen area to

$$A = \frac{4}{3} \cdot \frac{Q}{\mu v}$$

in which μ represents the coefficient of contraction, the value of which is generally taken as 0.70. This expression may be simplified and written

$$A = 2\frac{Q}{v}.$$



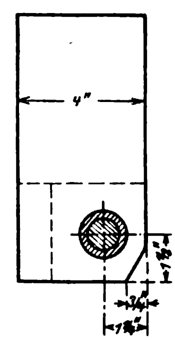
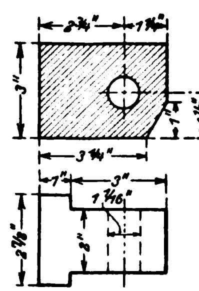


Fig. 193. Details of Screen Construction.



It is well to observe that the width of screens is governed by the length of the power plant or forebay, and the remedy of increasing the depth introduces another evil: the difficulty of efficient cleaning.

For this purpose, screens are generally inclined to an angle of 45 or 50 degrees with the horizontal. At the upper end, a bridge about 3 feet wide affords room for a workman to remove all material that is being held up. It is obvious that the maximum depth must not be beyond safe reach, and the only remaining solution of increasing the effective area is to place the screens before the piers which separate the bays, instead of in between. In this manner the whole length of forebay is available for clear space.

If h = the elevation of water level in the forebay; and
α = the angle of inclination of the screens with the horizontal,

the length of the section may be determined from the relation

$$h = l \sin \alpha$$

which gives

$$l = h \operatorname{cosec} \alpha .$$

If the screens are vertical $\alpha = 0$, $\operatorname{cosec} \alpha = 1$, and $l = h$. To anyone who takes the trouble to examine closely the arrangement of a system of screens, it can scarcely fail to be evident that either inclined or vertical, they are a source of trouble. The space between the bars may become clogged with ice, stone, wood, etc., thereby reducing the effective area, and increasing the velocity of flow.

This increase in velocity, besides being a loss of head, really attracts and accumulates more stones and drift, and requires continuous work to keep the rack clean. According to M. A. Boucher, the plant of Chevres, in France, has at one time during high water, necessitated the services of 70 men solely occupied in removing material held up by the racks; and at the Rheinfelden plant in Germany, cases were recorded in which to prevent the shutting down of the station, each bay required the attention of 5 men, continuously engaged in diverting considerable quantities of material carried to the forebay by the rapid waters of the high water season.

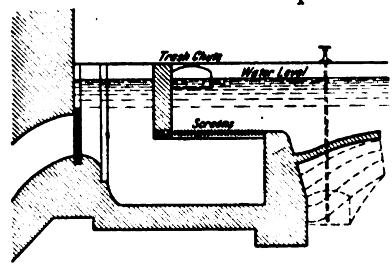


Fig. 194. Arrangement of Horizontal Screens.

Such disagreeable experiences, and the continuous possibility of their recurrence, have led to an ingenious arrangement shown in fig. 194 which originated in Germany.

The screens are laid horizontally. The water flows through them perpendicularly to the river bed and floating matter is directed to a chute provided for the purpose. Heavy material as stone, gravel, etc., carried along by the force of the current, accumulates in a sump hole leading down to a sluice gate especially provided in the forebay masonry. The screens themselves require no attention at all and the scheme although increasing somewhat the first cost of a plant, offers the best possible solution of the screen problem, and removes all possibility of trouble and expense during the operation of the plant.

118. The Wheel Pit.

The chamber in which the wheel is set, is called the wheel pit, and is generally of the open type for low or moderate heads (36 to 50 feet). In the case of high heads, closed flumes are generally used, the result being a smaller percentage of loss due to friction. It will be noted that in plants with open or closed wheel pits, 20 feet of the head may be covered by the draft tube. The chamber leading to the wheel must be designed so as to increase the velocity of the water as it approaches the turbine, in order that the loss due to a sudden change in velocity may not be too great. Losses from head- to tail races, resulting solely from wheel pit and draft tube friction, may vary from 4 to 12 %, the loss being highest in low head plants. This may be explained by the fact that in high head plants, the wheel pit takes the form of a closed conduit, directly connected to the head race, and in which there is little change in the friction of that part of the pipe replacing the wheel pit. It is desirable that each turbine be placed in a separate pit, so that one or more wheels may be shut down for repairs or other causes, without interfering with the operation of the plant. Therefore it is necesssary to separate the wheel chamber from the head race by means of suitable gates. Experience shows that a uniform loss of one foot may be allowed in the design of wheel pits for any head.

Proper foundation must be given to the turbine in order to prevent its settling out of alignment, and to reduce the vibrations of the machine. Undue friction will thereby be eliminated and a better efficiency obtained.

119. Draft Tubes.

Relatively little is known of the conditions governing the design of draft tubes, also called suction tubes. Theory indicates a possible length of 34 feet as the suction limit for draft tubes, but the balancing height of water in feet cannot be greater than:

$$\left(34 - \frac{v^2}{2g}\right)$$

where v is the velocity of the water in the tube; if the vertical length is greater than that as determined by the above expression, a greater loss of head occurs. Besides, there are other losses to be taken care of: entrance loss; friction loss; losses due to change of velocities within the tube, etc.

Table L gives the interdependent relations between the diameter of draft tubes and the corresponding draft head, the latter being the vertical distance between the center of the shaft of the turbine, and the level of the tail race water. Draft tubes are preferably made conical in shape so as to reduce gradually the velocity of the water, the upper end being of the same area as the discharge opening of the turbine casing. When the tubes are set vertically, they must not form a right angle at the bottom, as a strong bend causes an appreciable loss of head. The velocity of the water at the lower end must not be less than 2 or 3 feet per second to prevent air bubbles rising in the tube.

Table L.

Diam. of Draft Tube, in Feet	Draft Head in Feet	Diam. of Draft Tube, in Feet	Draft Head in Feet
0.5	32.5	8	14.5
1	30.0	9	13.0
2	27.5	10	12.0
3	25.0	11	11.0
4	22.5	12	10.5
5	20.0	13	10.0
6	18.0	14	9.5
7	16.0		

Draft tubes are usually built of riveted steel plates, thoroughly water tight, and in order to seal them against the inrush of air, they are dipped below the level of the tail water, this being done with due consideration to the lowest tail water level. Short and small draft tubes may be dipped from 6 to 12 inches, large tubes up to 24 inches. In case these are imbedded directly in the concrete, their arrangement is mostly controlled by rock excavations and elevation of tail water at flood times. The efficiency of a wheel is somewhat influenced by its position with respect to the draft tube. It is generally admitted that for horizontal wheels, long draft tubes are necessary, although the best length, shape, inclination, etc., is a matter open to conjecture. In general they must

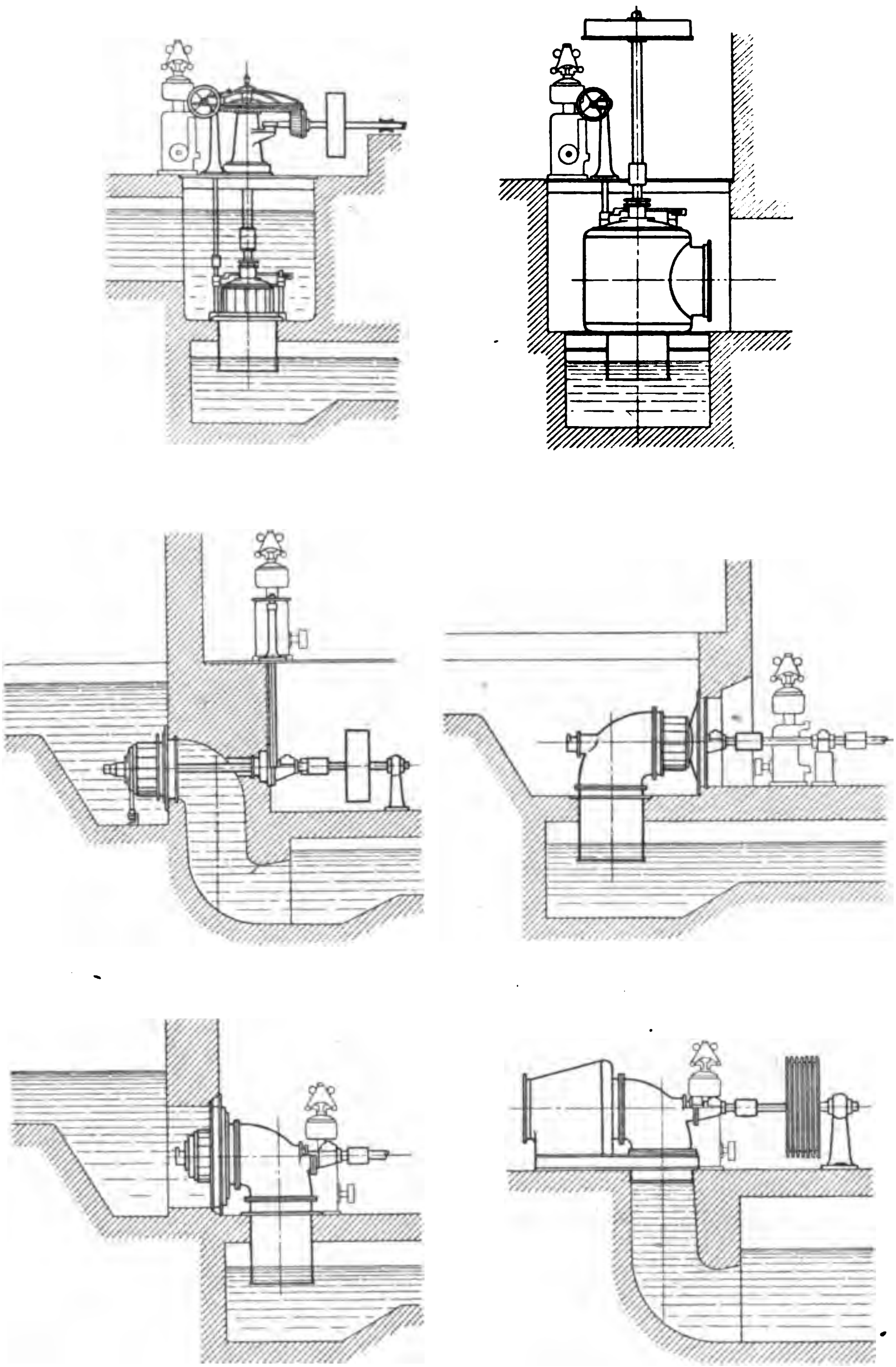


Fig. 195. Typical Francis Turbine Installations.

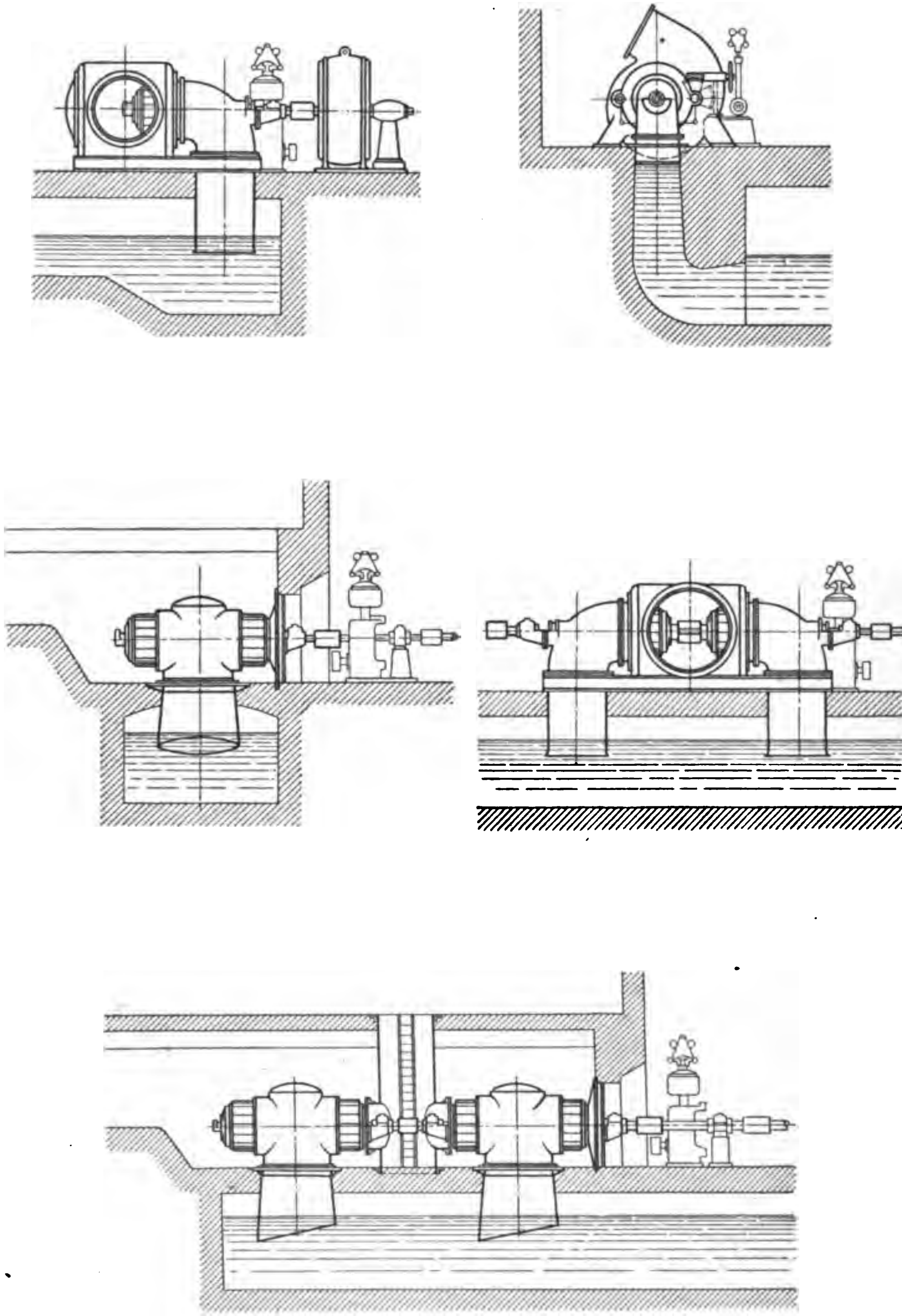


Fig. 196. Typical Francis Turbine Installations.

be as free from turns as possible, and it must be borne in mind that the greatest efficiency of the wheel will be obtained by designing and placing the tube in an inclined position with its cross-section area gradually increasing from wheel to outlet.

120. Headrace and Tailrace.

Generally speaking, headraces should be comparatively narrow and deep, especially where they are of considerable length and a large quantity of water is to pass through them. The velocity of the water in head- and tail-races should not exceed 2 to 3 feet per second.

When the water is conveyed to the turbines by means of penstocks, the intake of these must be well submerged to prevent the air from entering them, as the water level of the reservoir is lowering. The velocity of the water in penstocks has no definite limiting value, some engineers advocate velocities of 3 to 4 feet per second, although in the Schaghticoke plant for instance, the velocity reaches nearly 12 feet per second.

The intake should be located in such a way as to prevent the sand in suspension being carried into the headrace, and in order to keep out debris, ice and logs, floating booms or submerged arches are generally installed, their direction being such as to deflect the floating matter and carry it down stream.

A novel arrangement, omitting the use of head racks is illustrated in fig. 197, and is due to the French engineer, A. Crolard.

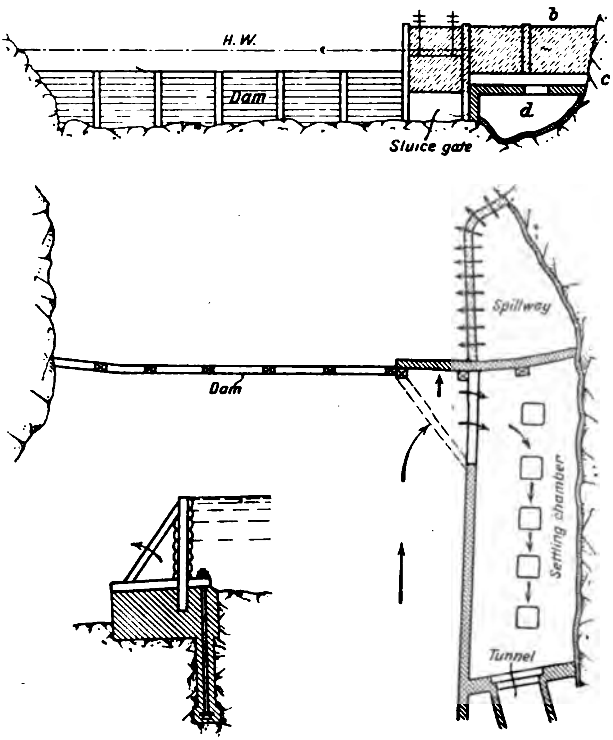


Fig. 197. Crolard Type of Headrace.

Floating debris are retained by the boards *b*, as the water passes through the small submerged opening *c*, and the suspended matter is passed through the sluice gates. Finally the sand settler *d* is so constructed that it can be flushed out from time to time.

Tailraces are usually wide and deep and there must be at least a dead water depth of 2 feet for small turbines, and of at least 4 feet for large turbines. With tailraces of such construction, as soon as the water is discharged from the turbines, it will push out or displace the dead water in the race, thus preventing a loss of head.

In the design of tail races it must be borne in mind, that what is needed is an easy discharge of the water, a condition that can be obtained by the proper use of deflecting walls or booms, so as to prevent the possibility of the water backing up around the draft tube.

The different methods of installing Francis turbines are illustrated in figures 195 and 196. The general outlines will be found very useful by the designer of power plants.

121. Alternating Current Generators.

For the long distance transmission of electrical energy, alternating current generators are exclusively used, although in some recent developments, the Thury direct current series system has been employed.

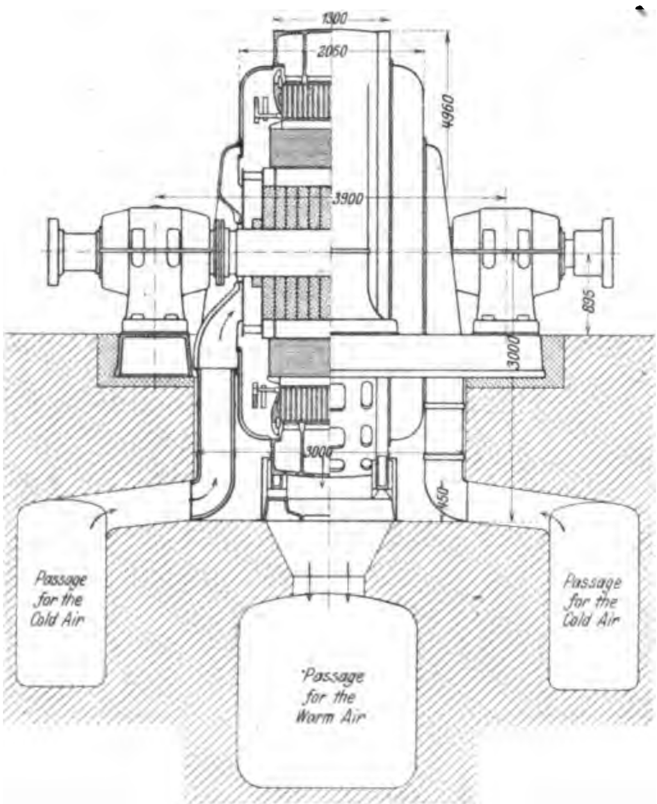


Fig. 198. Cooling Arrangement for Electric Generators.

Modern types of alternators consist in machines in which the armature is the moving member but of late, another machine has been developed in which the armature is stationary. The latter system has its advantage in that the stationary parts can be insulated with facility for high voltages, and, as no collecting system is necessary, any amount of current can be cared for. Generators of this type have been built for voltages up to 30,000 volts, simplifying the station wiring and eliminating step up transformers, a thing which is obviously advantageous, if the transmission line voltage is to be the same as that of the generator.

However, if the distance of transmission is so great as to require too high a voltage to be supplied directly from the generator, it is better to use a low voltage generator, say 6600 to 11,000 volts, in conjunction with step up transformers. It must not be overlooked, though, that the generator voltage should not be too low, as conditions may arise in which part of the line voltage is electro-statically impressed on the generator

winding, increasing the insulation stresses to a dangerous degree. If simplicity of station wiring is desired, and if the transmission line voltage does not exceed 20,000 volts, it is preferable to wind the generator for this voltage. In other cases, it seems conservative to use step up transformers, although in one case at least (Manojlovac Plant, Dalmatia) the 42 cycle, 3-phase current is transmitted at 30,000 volts, which is the generator voltage.

The construction of generators of very large outputs and correspondingly high speeds requires that special attention be paid to the cooling of the machines through special ventilating arrangements, and the rotating parts must be dimensioned so that in case of the prime mover getting out of control, these parts would be capable of resisting the additional stresses due to increase in speed.

These generators differ in their external appearance from those for lower outputs and speeds, in their enclosed form of construction which is essential for the ventilating arrangements. For small outputs cooling air is drawn direct from the machine room and exhausted into the same room; in the case of larger machines, in order to avoid too great a noise due to the ventilation and to prevent accumulation of dust and fluff in the machines, air is drawn from outside through special passages, intermediate filters being sometimes used, and is also exhausted outside the building.

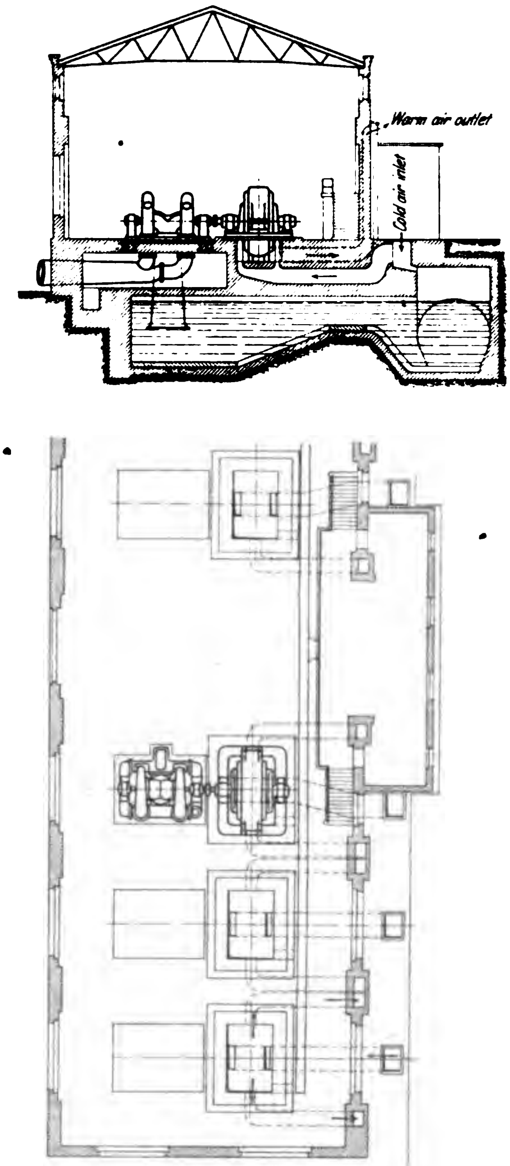


Fig. 199. Ventilating Ducts for Electric Generators.

Fig. 198 shows a generator of the A. E. G. type, with cooling arrangement. After the cooling air has passed over the heated parts, it is led through wide openings in the lower part of the housing to the air exit passage situated below the machine.

The plan and elevation of a generating station in which the air is drawn in and exhausted through special passages, are shown in fig. 199. Each machine has its own inlet passage and two outlet passages. This

arrangement possesses the advantage over that in which all machines draw their air from one common channel in that the separate machines have no influence no each others' ventilating arrangements.

122. Switchboards.

Switchboards are used to collect the generated current for the purpose of its control, measurement and distribution.

There are two general types of switchboards: the *direct control board,* in which the apparatus is located either on or near the operating panels, and the switches are operated by hand; the *remote control board,* in which switches and bus-bars, are installed at some distance from the operating panels, and the control is effected either by mechanical devices, such as bell cranks, rods and gears, or electrically by solenoids or motors. In a few installations, the remote control system is operated by compressed air. Direct control boards are generally located on a gallery overlooking the generator room.

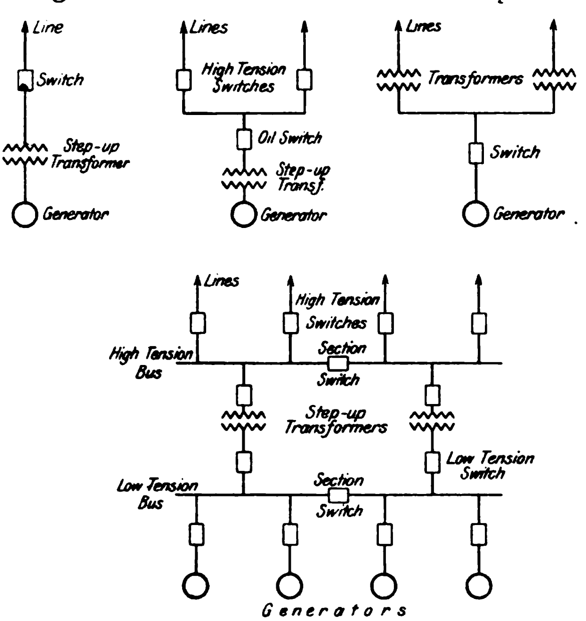


Fig. 200. Methods of connecting Stations to the Line. (With Transformers.)

No general rules can be laid down for the location of switchboards except that they be so placed that the wiring from the machinery to the switchboard be not excessive in length, and when installing a switchboard, provision must be made for future extension. The connections on the back of the board are made with insulated wire for the control and auxiliary windings, whereas bus-bars are used for larger currents. These bars are generally made up of rolled copper from $\frac{1}{4}''$ to $\frac{3}{8}''$ thickness, to 2 inches in width. They are designed on the basis of a maximum current density of 1000 amperes per sq. in. of cross section. Contact surfaces between bus bars must be designed for an allowance of 100 to 200 amperes per sq. inch of area.

There are three methods of support for control switches and instruments: the *panel type,* in which the control switches and instruments are mounted on vertical panels as in the direct control board; the *bench or desk board* arrangements, in which the control switches are mounted on panels that are horizontal, or slightly inclined to the horizontal, the measuring instruments being mounted on vertical panels in front of the bench board. Miniature dummy bus-bars are mounted on the bench, and

the positions of the main switches, whether open or closed, are indicated by means of signal lamps placed near the control switch. The *pedestal or column type* of board consists in that the control switches and rheostat hand wheels are mounted on pedestals. The indicating instruments are mounted on panels, or, as in some cases, on separate pedestals, so that the operator may readily observe them.

The methods of connecting stations to the line are shown in fig. 200 and 201.

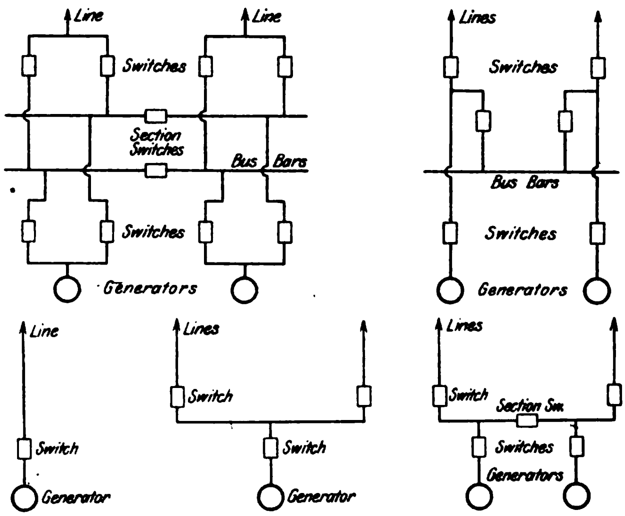


Fig. 201. [Methods of connecting Stations to the Line. (No Transformers.)

123. Circuit Breakers and Relays.

High tension circuit breakers are switches equipped with an automatic opening device which opens a circuit under predetermined electrical conditions of abnormal nature.

Their object is the protection of generators and transformers and they are built for conditions of underload, overload, and no-voltage or reversed current. They may open in air or in oil, the latter being usually the case with high voltage circuits. When circuit breakers are connected to large capacity and high voltage circuits, they are usually of the remote control type, being actionned by some sort of power. In such cases, automatic operation is obtained by means of relays.

Overload relays are generally built in single phase units and for either instantaneous, definite time limit or inverse time limit operation.

Overvoltage relays are used, as the name implies, to prevent damage to generators and transformers from excess voltage in the circuit. The reversed current relays are a protection against the reversal of flow of energy, as may be caused by the use of synchronous machines.

124. Automatic Voltage Regulators. Tirrill System.

The Tirril regulator varies the terminal voltage of the exciter of an alternating current generator by maintaining a constant resistance in the main exciter circuit, so that under variations of load, power factor or

spead, the voltage of the alternator will remain constant. The advantage of the Tirril regulator consists in the fact that it maintains a constant pressure by automatically overcompounding the generator either at the station bus-bars or at any point of the distributing system.

Regulation is accomplished by altering the terminal pressure of the shunt exciter while a constant resistance is maintained in the exciter circuit of the alternator. A short circuiting contact, connected in parallel with the shunt regulator of the exciter, is alternately opened and closed by means of a lever which oscillates several hundred times per minute. The average value of the shunt current flowing through the short circuiting contact will increase in the direct proportion of the difference in the time taken in closing over the time taken in opening; and the terminal voltage of the exciter will rise with the current.

The exciter voltage can be varied between a given minimum value with the generator running on no-load, and a maximum value at full load. The field magnet regulator of the generator is either short-circuited or adjusted at a constant low value; it must not be entirely omitted, since it serves as a reserve when it is desired to parallel a generator running on no-load with a fully loaded machine in cases where both machines obtain their exciting current from the same source.

When two or more generators are running in parallel, it is not as a rule permissible to regulate each generator by means of a separate apparatus on account of the fact that very small differences in the adjustment of the separate regulators cause considerable differences in the exciter currents of the separate generators, and the unequal division of the wattless current components amongst the individual generators; equalising currents are also set up.

When generators with different outputs are working in parallel with one another, only the large units are regulated in accordance with this system. As the small generators are not influenced by the Tirrill regulator when running, the excitation of these latter machines is to be regulated by hand from time to time, as desired current distribution will demand.

125. Lightning Arresters.

Transformers, switching stations and transmission lines must be protected from lightning or other high potential surges. This protection is accomplished by means of lightning arresters, of which various types have been developed.

The *horn lightning arrester* is based on the principle that a short circuited arc once started at the narrow gap between the horns, rises along the members and gradually blows itself out.

Fig. 202 illustrates a horn lightning arrester as adopted by the American River Electric Co. It is installed on a 40,000 volt transmission line. and made of galvanized iron gas pipe, mounted on insulators on a pole construction, leaving a gap of 2.25 inches.

The ground is effected through a 25 gallon water tank containing a film of oil on top to prevent excessive evaporation.

The horn type of arrester is not much favored in systems which

require reasonable continuity of service; in case of a discharge over the horn, motors and converters usually drop out of step, and the generators break their synchronism, so that the whole system must be started up again.

For this reason, horn gap arresters are preferably used in conjunction with some other type.

Fig. 203 gives the setting of horn gaps for different voltages.

The *multigap arrester* consists of a large number of spark gaps in series with each other, between metal cylinders. It is described as follows by F. Creighton[1]: "The mechanical parts of a multigap lightning arrester consist of a number of brass cylinders spaced about $\frac{1}{32}''$ (0.8 mm) apart, in combination with various values of resistance. In multigap arresters of the highest efficiency, the resistances are used in shunt with groups of gaps, fig. 204. In the first path of the lightning, there is placed a high resistance R, and the minimum number of series gaps (group G) which will just extinguish the arc for the value of dynamic current that would flow through the resistance. There is a second path for the lightning passing through a medium value of resistance M, and the

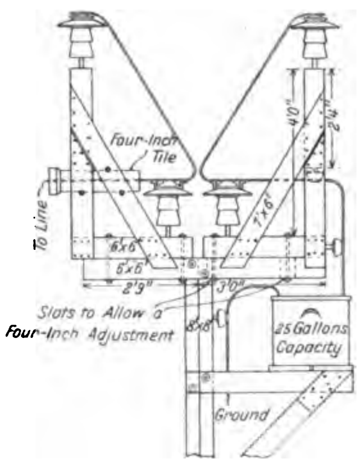


Fig. 202. Construction of a Horn Type Arrester, American River Electric Company.

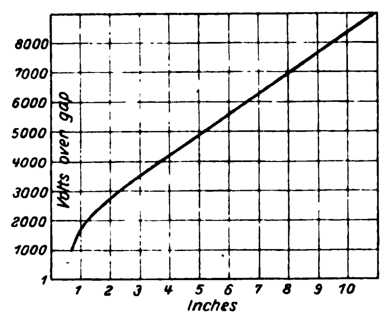


Fig. 203. Curve Showing Setting of Horn Gaps.

minimum number of gaps $G_2 + G_1$ necessary to extinguish this greater value of current. There is a third path for the lightning through a low resistance r, and a still greater number of gaps to ground $(G_3 + G_2 + G_1)$. There is still a fourth path for the lightning to ground, in which there is no series resistance, but a great many series gaps $(G_4 + G_3 + G_3 + G_1)$. The object of this combination is to have resistance in series where it is permissible on light strokes of lightning, but have it possible to cut out automatically more or less of the resistance in accordance with the demand of the severity of the lightning stroke."

The theory of spark potential of the multigap arrester was given by Dr. Steinmetz in 1906. When in a lightning arrester, all of its gaps do not begin to spark continuously, the action is successive. The gap nearest the line sparks and by its oscillation at nearly one billion cycles per second, transmits the spark to the second gap. The oscillation of the second gap transmits the impulse to the third, and so on, throughout the total number of series gaps.

[1] Protective Conditions of Electrical Apparatus in America. E. E. F. Creighton. Congresso Internazionale delle Applicazioni Elettriche. Turin 1911.

The arrester is very sensitive to high frequency surges, but is somewhat proportionately less sensitive to lower frequency surges, therefore the arrester has no fixed spark potential. A concrete example will illustrate this variation. If 32 gaps each one $\frac{1}{32}''$ (0.8 mm) long, are placed in series, the sum of the gaps is equal to one inch space (2.54 cm). If high frequency is applied to this series of gaps with a variable needle gap in parallel, it is found that the equivalent-needle-gap may not be greater than $\frac{1}{4}''$ (6.35 mm). If however a uni-directed impulse of potential is discharged through the arrester, it is found that the equivalent-needle-gap approaches $2''$.

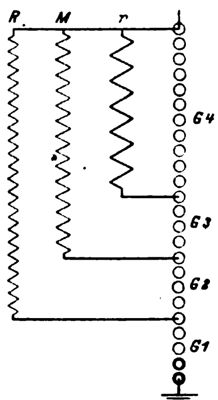


Fig. 204. Multigap Arrester.

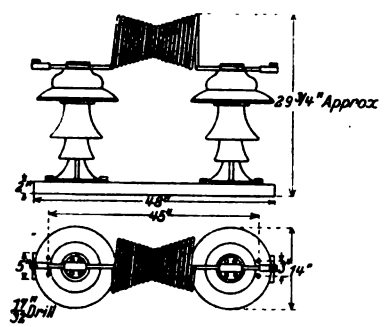


Fig. 205. Hour Glass Type of Choke Coil.

Choke coils, fig. 205, consist of a number of turns of wire in series with the transmission line and are preferably used in connection with lightning arresters as they offer the advantage, if installed in the leads to a generator or transformer, to reduce the potential between the end turns, and, on large electrical systems, they limit the current in cases of accidental short-circuits.

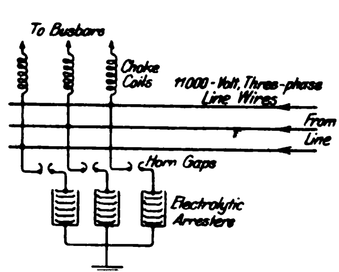


Fig. 206. Connection Diagram for Electrolytic Lightning Arrester. 11 000 Volts.

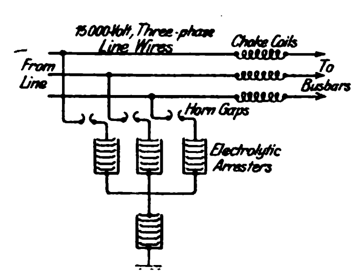


Fig. 207. Diagram of Connections of 15000-Volt Electrolytic Lightning Arrester.

The *electrolytic lightning arrester* consists of nested aluminum trays filled with electrolyte and submerged in oil.

The horn gaps which are between the arresters and the line break down on over-voltage, and if this voltage amounts to approximately more than 330 volts per tray a free passage through the film from tray to tray

and thence to the ground is provided. When the voltage drops to normal, which is below the critical voltage of the tray, the apparent resistance of the film reasserts itself and the flow of the current is cut down so that the horn gaps disrupt the arc and all further flow of current ceases.

The nested trays are placed in oil filled, welded steel tanks, which for 22000 volts and above are built with cast covers and outdoor terminals so that the entire arrester can be placed outdoors. The complete arrester consists of four tanks mounted on an insulated platform for ungrounded neutral service, and three tanks grounded for grounded neutral service. For 13200 volts and under, the arrester is arranged for indoor service only as the horn gap setting is so small that much more effective service is obtained when the arrester is housed. For the lower voltages the general arrangement of trays is the same, but a smaller number of tanks is used.

Since when the electrolytic type of arrester is allowed to stand without current passing through it, the film on the plates dissolves, and it is necessary to pass current through the arrester periodically, in order to keep the film in the best possible condition.

Figs. 206 and 207 are diagrams showing the connections for electrolytic lightning arresters as installed for the Pittsburgh Railway System, and fig. 208 shows the construction details for such arresters.

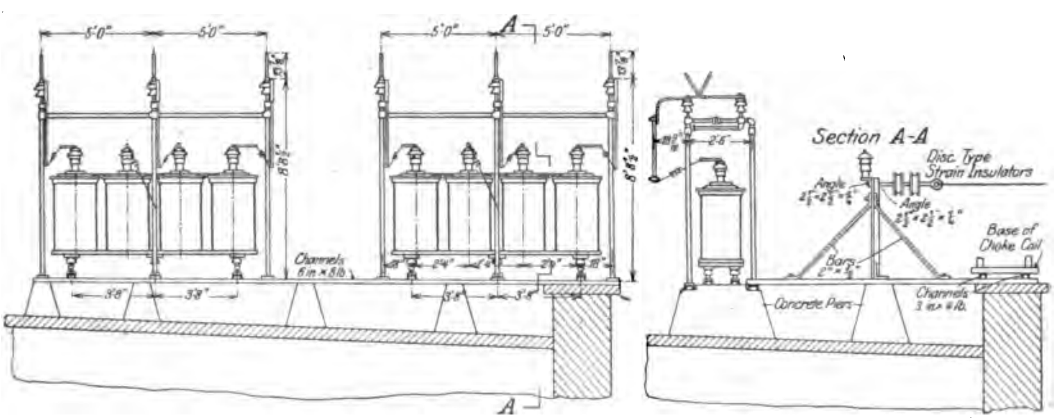


Fig. 208. Diagrammatic Drawing showing Steel and Constructional Details of Frame for Electrolytic Lightning Arrester.

In regard to location of lightning arresters it has been the practice to place choke coils and multigap arresters inside the station, while horn gap and electrolytic arresters are generally located outdoors.

126. Transformers.

At higher voltages, step-up and step-down transformers are always used, forming part of the high tension circuit.

The electrical transformer is the simplest alternating current apparatus and its object is to transform *a. c.* energy at one voltage to *a. c.* energy at another voltage, either higher or lower. Fundamentally,

the apparatus consists of a magnetic circuit of iron interlinked with two electrical circuits, usually of a large number of turns. In these, the currents are inversely proportional to the voltages of the windings.

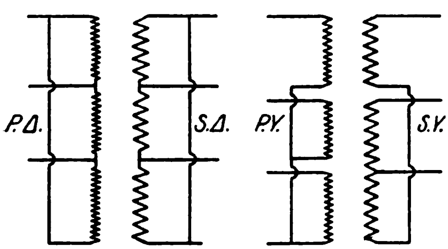
A three-phase transformer is made up of three single-phase transformers combined into a single piece of apparatus, and the magnetic circuit is common to the three phases.

In long distance transmission lines, three-phase current is preferably used and the transformation system may be made by connecting the primary either in Y or Δ and the secondary either in Y or Δ. The relations between voltage per transformer winding and voltage between lines and between current per line and current per winding, are as indicated in Table LI for Y and Δ connections.

Table LI.

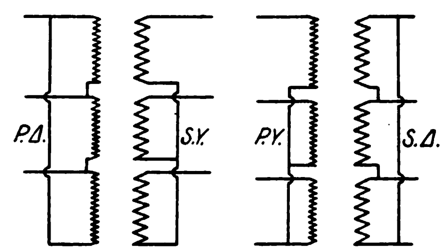
Connection	E between lines	E per winding	I per line	I per winding
Y	E	$0.58\ E$	I	I
Δ	E	E	I	$0.58\ I$

It must not be forgotten that when more than one transformer is used, each must have a capacity equal to the power required to be transformed by it, divided by the power factor of the circuit which it supplies.



a b

Fig. 209 a.



a b

Fig. 209 b.

In fig. 209 a, 209 b, 210 are shown the usual connections utilized for group of single phase transformers for three-phase transformation. In (a), the primary windings and secondary are both connected in Δ. In (b) both primary and secondary are connected in Y. In fig. 209 b (a) the primaries of the transformer are Δ connected while the secondaries are Y connected, In (b), the primaries are Y connected while the secondaries are connected in Δ arrangement. When the transformers are to be connected as per fig. 209 a and 209 b the terminals must be correctly joined according to their polarities, or the voltage phases will be unbalanced in the secondary circuits. When two transformers only are used, the primaries may be T-connected and the secondaries T-connected, fig. 210; or two transformers on a three-phase circuit may be connected in Y or open delta

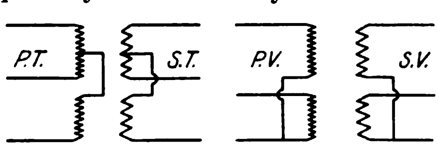


Fig. 210.

connection. Lastly, the primaries may be V connected and the secondaries Y connected; this is also called the unsymmetrical Y connection, because this arrangement does not give a balanced three-phase system on the secondary.

An important problem which arises in connecting the three phases of the alternator is the question of harmonics, if the machine is wound in the Δ arrangement or if the phases are connected in Y.

The capacities of each transformer for Δ and Y connections, must be equal to one third of the total load in kilovolt amperes. This for any-power factor is $E_Y I = E I_1 = \frac{1}{\sqrt{3}} E I$. The three required transformers have then a total capacity of $3 \frac{1}{\sqrt{3}} E I = \sqrt{3} E I$.

If the windings are Δ connected, they will produce full line voltage and carry a current $\frac{I_L}{\sqrt{3}}$, but when Y connected, they will produce a voltage of $\frac{V_L}{\sqrt{3}}$ and carry full line current. In T connection, the coils carry the same currents as the line wires, but the voltage of one transformer is only 0.867 time the line voltage.

The four combinations of Y and Δ connections are given in Table LII and may be used on any bank of transformers.

Table LII.

Connections		Low Potential Volts between Lines	High Potential Volts between Lines[1]
Δ	Δ	E	$u E$
Y	Y	E	$u E$
Δ	Y	E	$1.73\, u E$
Y	Δ	E	$0.58\, u E$

[1] u is the ratio of transformation of primary to secondary.

The relative advantages of Y and Δ connections are summed up as follows:

With a $\Delta - \Delta$ system, it has been the custom to delta-connect the transformer primaries to permit one set of coils on a three-phase transformer, or one of a bank of three single phase transformers, to be cut out of the circuit in case of trouble. It is then possible to connect the remaining two in open delta, thereby realizing the approximately 58% of the output of the bank with the same heating. However, if one line becomes grounded, the voltage strain becomes $1.73\, E$, which may be transferred to the secondary circuit on which the generator is connected.

With $Y - Y$ connection, the neutral is unstable and an excessive voltage strain on one phase is possible, unless the neutral is grounded.

With $\Delta - Y$ (step-up) connection, the neutral on the high tension side may be grounded. The voltage strain on any transformer is limited to 58% of the line voltage. The neutral can be used as a third conductor for operating 2 transformers of a bank if one is damaged. With the $\Delta - Y$ connection, however, there is a possibility of resonance, disturbances on telegraph and telephone lines. An accidental ground is a true short-circuit for the grounded phase.

In $Y - \Delta$ connection (step-up) the neutral is unstable with unbalanced load, but stable with balanced load.

As for a grounded neutral, the advantage lies in the fact that the system maintains a static balance.

The choice between ungrounded and grounded neutral, resides in the fact that the first one is more reliable and for Δ connections, preferable; the grounded neutral is advisable when cheapness is the object sought. In the latter case, Y-connections are recommended.

As the wave shape of the generated *e. m. f.* of most alternators contains higher harmonics, and a third harmonic is present in the magnetizing current of transformers that operate at high magnetic densities, the result is that in Y-connected apparatus, if harmonics are present, they produce a wabbling neutral.

If the neutrals of 2 different machines are grounded or joined, high frequency current flows in the neutral.

In Δ connections, the distorted third harmonic and neutral are made stable at low cost.

The commercial efficiency of transformers varies from 95 to 98% at full load.

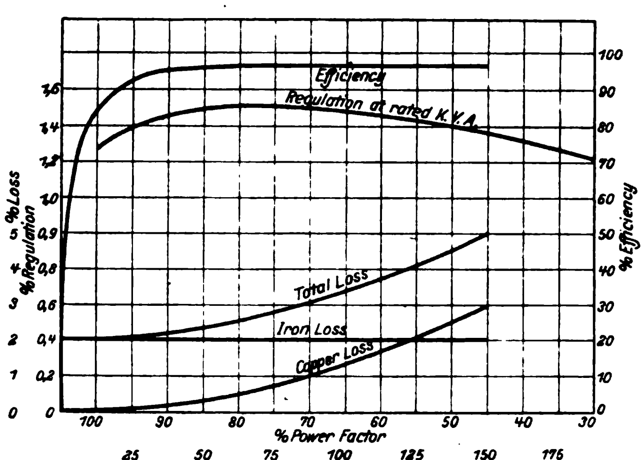


Fig. 211.

Fig. 211 is a diagram giving the losses, efficiency and regulation of a 100 Kw 12000 volts 50 cycles Westinghouse air-blast transformer. These characteristics are representative curves of all transformers. —

The relative advantages of three-phase and single phase transformers may be summarized in stating that moderate sized transformers, compared with single phase transformers of the same capacity, have the advantage

of lower cost, higher efficiency, reduced floor space, less weight, simpler wiring, smaller freight and erection charges, although they present the disadvantage of greater cost if spare units are used. When the capacity of the three-phase transformer or of the bank of single-phase transformers reaches 10,000 K. V. A., the use of single phase units is preferable, as they are easier to handle than the equivalent three-phase unit, although the Great Western Power Company has in successful operation 10 three-phase 10,000 kw transformers for the step-up end of its 154 mile 100,000 volt transmission line.

127. Transformer Cooling.

The temperature of a transformer during operation is a matter of great importance. Small transformers have comparatively large surface as compared with the heat produced, so that they cool easily by self radiation. The average transformer of 50 to 75 kw capacity is cooled by being immersed in oil; they are of the self-cooling, oil-filled type; the heat generated in the coils and iron is given up to the oil and the oil in turn dissipates the heat to the air. In larger transformers, the losses and consequently the temperature increase more rapidly and special means of cooling must be provided.

The *oil-cooled transformer* offers several advantages. The large volume of oil absorbs considerable heat and therefore the temperature rises slowly. Another advantage consists in the high insulating qualities of the oil itself, besides possessing the power of self-repairing any break in the insulation, and preventing oxidization by the air. The oil in the transformer is cooled either by its natural gravity circulation, or by means of submerged coils through which water is circulated. The latter is a most effective method of cooling and is very convenient for water power plants, the supply and pressure being at hand. As the water absorbs a high percentage of the total energy loss, it is unnecessary to provide a case with a large cooling surface. This method should, therefore, be adopted for large transformers, when a good supply of cold water is available. If this is not available, the same water can be used continuously with the use of an outside cooling arrangement and a circulating pump. If, owing to a temporary break-down of the pump, the water supply is stopped, no great danger may be apprehended, as the enormous thermal capacity of the oil and the water in the coil prevents any large rise in the temperature for a considerable time. On no account should a joint be made in the coils inside the case, as this leads to a liability to leakage, the slightest trace of moisture in the oil reducing its dielectric strength very considerably. With regard to unassisted oil cooling, this is much more effective than any type of air cooling, as the thermal conductivity of iron to oil is so much greater than to air. The cooling by air blast, although effective, necessitates the use of blowers and air filters, and further, if for any reason the blast is stopped, the temperature of the transformer will quickly rise to a dangerous degree. Natural cooling by air is simple and economical, but this method can only be applied to transformers of small size, as the energy loss, for the same induction and

frequency, is proportional to the cube of the linear dimensions, whereas the cooling surface is only proportional to the second power, and, therefore, the heat to be dissipated per square inch of surface rises rapidly with increase in size, producing thereby a higher temperature rise of the transformer.

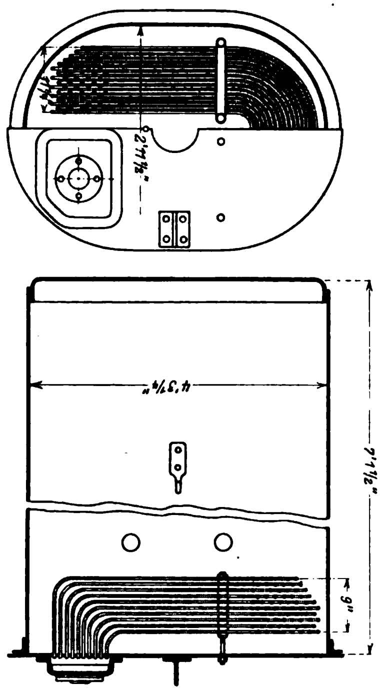


Fig. 212. Box for Grannige 720-kw Transformers.

Fig. 212 shows the arrangement of the cooling coils of the transformers used at the Grannige plant in Sweden. Instead of having one or a few circulating pipes of large diameter, or a water jacket, the pipes consist of a number of copper tubes of small diameter placed in parallel, like a water tube boiler. In this construction the tubes are expanded or sweated into headers which form one side of the distribution boxes for the inlet and discharge of the circulating water. The entire system of cooling tubes is placed at the top of the transformer box over the transformer itself, and is attached to the transformer box lid, so that when the lid is removed the coils are removed with it. By employing this construction an enormous cooling area can be put into a very small space, practically the only limit to the room required for the cooling coils being the amount of space which must of necessity be left between the tubes in order to obtain good circulation of the oil. Moreover the cooling coils are at the top of the box where the oil is hottest, and hence in the best and most effective position. A very complete arrangement is provided for tapping out the oil, boiling

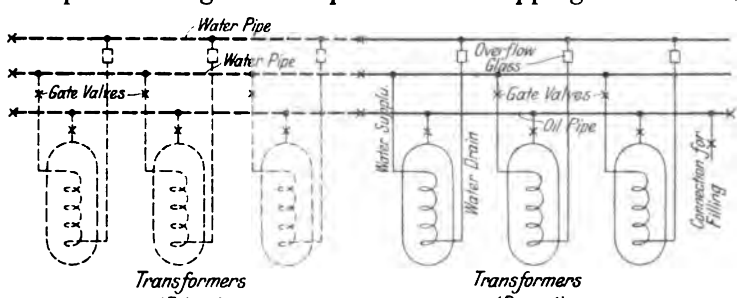


Fig. 213. Water and Oil Piping Diagram for Transformer Cooling.

and refilling. It consists of a system of pipes to and from the transformers, connected with oil-boiling tanks. The use of oil-cooled transformers has of recent years become general in Sweden, on account of the larger units now employed and the high voltages used in the transmission of energy.

Fig. 213 shows a water and oil piping diagram, and fig. 214, shows the method of cooling circulating water as used at the Molinar Plant, in Spain.

The amount of water necessary for cooling the oil depends on the temperature of the incoming and outgoing water. Theoretically each kilowatt loss will give up 56.9 B. T. U. per minute, or, in other words, 56.9 pounds of water are raised 1^{0} F. It is estimated that one gallon of water per minute is required per kilowatt loss. In practice however, the amount of water required varies with the design, and the data as to the amount of water required can be obtained from the manufacturer.

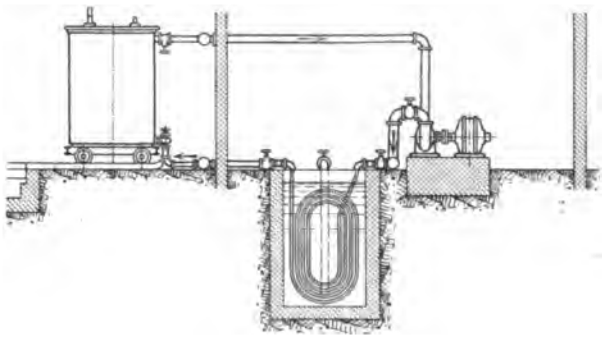


Fig. 214. Method of Cooling Circulating Water for 6750 K.V. A. 6600/66,000-volt 3-Phase Siemens-Schuckert Transformer, Molinar Plant, Spain.

The water-cooled type transformer has been necessarily the most popular design. It has greater ability to carry overloads than has the air blast type, and can be wound for any voltage. Such transformers have been manufactured in sizes up to 6000 kw, single-phase and 10,000 kw three-phase.

The *forced oil cooled transformer* provides for the oil being cooled outside in a cooling system such as illustrated in fig. 215.

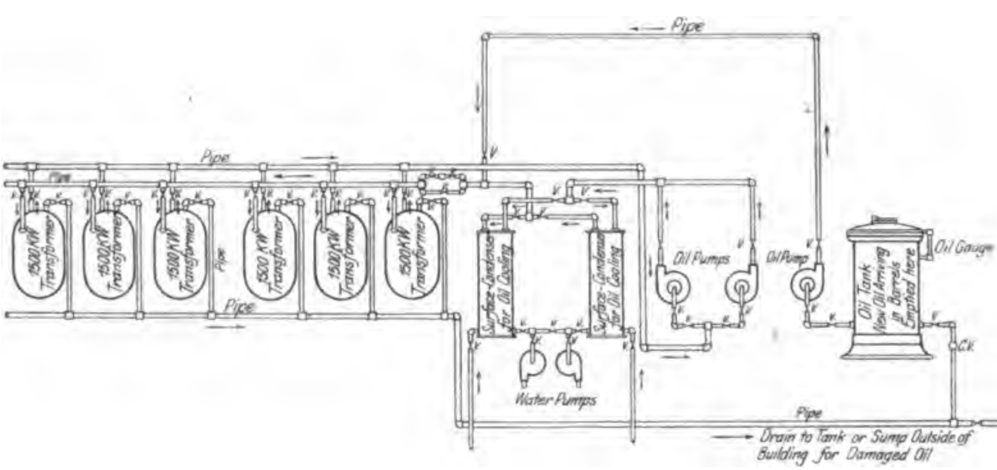


Fig. 215. Forced Oil Circulation for cooling Oil-Insulated Transformers.

It is observed that besides water pumps, a set of oil pumps is necessary although where sufficient head is obtainable the water pumps may be eliminated. The saving in cost of the complete equipment in using forced oil instead of forced water circulation may reach 25% for 10,000 kw units, but in small plants, the cost of the extra-cooling system will outstrip the reduced cost in transformers.

However, the use of forced oil cooling allows single transformer units to be manufactured for a rating as high as that of any single generator.

In the *air blast type*, the cooling is effected by means of a forced current of air circulating through the windings and core.

Where there are a number of transformers, they are preferably set over a common duct and supplied with air from a blower at either end

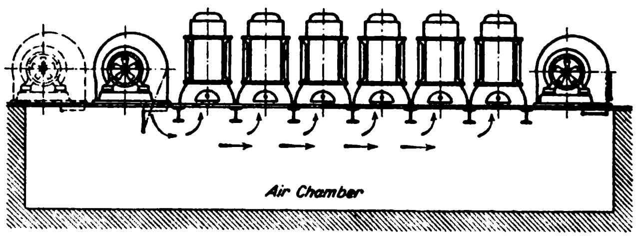


Fig. 216. Arrangement of Air Blast Transformers.

(see fig. 216.) The flow of air is regulated by means of two dampers, one of which is located at the top of the transformer and the other on the side of the frame. The first one regulates the air between the windings, the other controls it through the core. Fig. 217 shows a cross section of an air blast transformer and table LIII gives the volume and pressure of air required for transformers of various capacities.

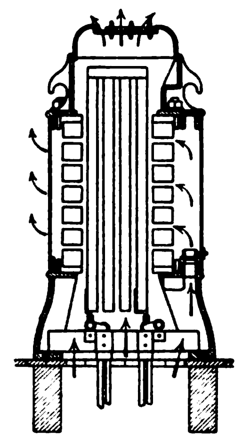


Fig. 217. Air-cooled Transformer.

Table LIII. *Air required for transformer cooling.*

Total kilo-watt trans.	Size of units kilo-watt.	Cubic feet air required per trans-former per min.	Cubic feet air required for all trans-formers per min.	Cubic feet air furnished by standard blowerset.	Oz. Press.	Freq. Mot'r.	Size blower, inches	Speed blower	Horse-power to drive blower full vol. and pressure
900	100	450	4,050	6,000	½	25	50	750	2.5
						40	50	800	
						60	40	900	
1800	200	900	8,100	8,000	⅝	25	55	750	4
						40	55	800	
						60	50	900	
2700	300	1125	10,125	10,000	⅝	25	55	750	5
						40	55	800	
						60	55	720	
4500	500	1625	14,625	14,000	⅝	25	75	500	5.5
						40	70	600	
						60	70	720	
6750	750	1875	16,875	20,000	¾	25	90	500	12
						40	80	600	
						60	80	600	
7500	1250	2800	16,800	20,000	¾	25	90	500	12
						40	80	600	
						60	80	600	

The theoretical quantity of air required for transformer cooling is determined by

$$Q = \frac{1.65\, W}{T}$$

where Q = cubic feet of air per minute,
W = watts lost,
T = temperature rise.

The temperature rise is usually one half the temperature rise of the copper windings.

The capacity of the blower-motor is determined by

$$P = \frac{p \cdot Q}{1200}$$

where P = capacity in horse-power
Q = cubic feet of air per minute
p = pressure in oz. (usually $\frac{1}{2}$ for small, $1\frac{1}{2}$ for large transformers).

The best oil for transformer insulating purposes is mineral oil, light or medium, having the following properties: it must not contain moisture; its dielectric strength to be good; its viscosity not to be high; it must not contain vegetable oil or resinoid material; it must be completely free from impurities. The most important factor in the quality of transformer oil is its purity; an impure oil may contain conducting material such as dirt, carbon deposit, metallic salts. In case of the use of the medium oil, which is rather dark colored, deposits are thrown down, tending to clog up the circulating ducts. This deposit is precipitated solely by the effect of heat. For large transformers especially of the water-cooled type, the heat is intense and much trouble would be encountered by the use of this oil; it is also preferable to use the light oil, usually known as mineral seal oil, which does not throw down deposit with an increase of temperature The detection of deposits in oil may be accomplished by drawing off some of the oil from the bottom of the transformer, where it generally accumulates on account of its greater density.

Concerning the effect of temperature upon the dielectric strength of oil, Mr. H. W. Tobey cites the case of a certain test in which a sample withstood an e. m. f. of 52,000 volts between two 0.5 inch brass disks 0.2 inch apart, when the temperature was 60° Centigrade, 45,500 volts at 20° Centigrade and 44,000 volts at zero degree centigrade. Upon congealing however, as the temperature drops below the freezing point of the oil, the dielectric strength increases with great rapidity, in some cases reaching a value from 60% to 80% higher than that just before the freezing point was passed. Transformer oil must not contain moisture, acid or sulphur compounds. The presence of moisture, which is absorbed very readily by the oil in rather small quantities, may be detected by testing a sample of oil drawn off from the bottom of the transformer. It is well known that the specific gravity of water is greater than that of oil, so that the moisture generally settles to the bottom. Such a sample when poured over some anhydrous copper sulphate, makes the latter turn to a bluish color. The moisture may be removed from the oil by boiling, the temperature being maintained at about 105° C, which is above the boiling point of water. This method however requires much time; in some cases

several days, depending on the degree of moisture. It may also be dried by baking in an oven, or applying the dehydrating method with metallic sodium.

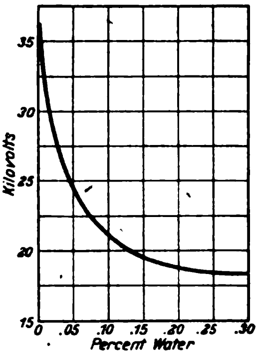


Fig. 218. Influence of Moisture on Dielectric Strength of Oil of Medium Viscosity.

Still another method of detecting moisture is to plunge into the oil a red hot piece of iron: if a sharp hissing noise takes place, water is present. The curves shown in fig. 218 and 219 have been determined by Mr. Tobey and they show the reduction in dielectric strength for gradually increasing percentages of moisture. Although it is true that water will settle on the bottom, nevertheless a certain percentage is always in suspension; this amount however should be very small, otherwise the oil must be removed. The curves show that even 0.03 $^0/_0$ moisture reduces the dielectric strength to three quarters. As to the best practice with either kind of oil, the medium grade is mostly used in self-cooled transformers, whereas the lighter oil gives better results in water-cooled transformers. Finally, transformer oil must be filtered frequently, and must be tested also for alkalis or acids by means of litmus paper.

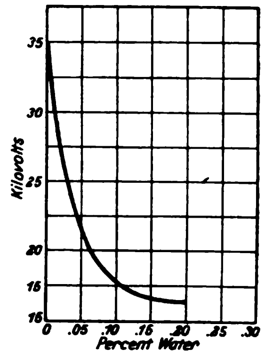


Fig. 219. Influence of Moisture on Dielectric Strength of Light Oil.

128. Substations.

In designing a substation, due consideration should be given to the building and to the immediate surroundings as well, the site being chosen with a view to ensure a direct and easy approach for the transmission lines.

In determining the dimensions of the building, the sizes must be such as to permit an increase in capacity of the plant, that is to say, room should be provided for installing additional machines together with the necessary additional switchboard apparatus.

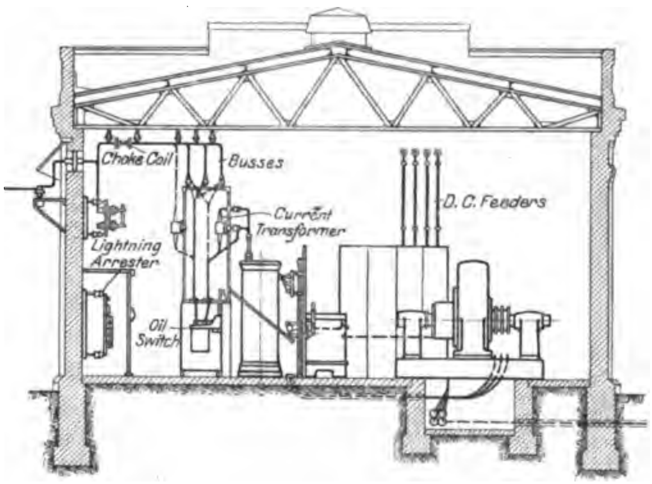
In transformer substations, transformers are usually located in fireproof compartments provided with iron rolling shutters. Fur the purpose of inspection and repairs, a track is run in front of the compartments, so that the transformers may be readily removed on a small truck, and shipped to the repair shop. In small substations, the transformers are not housed in compartments and are generally handled by overhead cranes.

Fig. 220 shows a plan and elevation of a small substation, with single phase, oil-insulated self-cooling transformers and hand-operated oil switch. This is the general type for 11,000 or 13,200 volt transmission line tension.

Where air-blast transformers are used, an air duct with doors, forming a vestibule, so as to permit one to enter without shutting off the blowers, should run under the space where the transformers are to be located.

If oil-cooled transformers are installed, it is good practice to provide a pit of sufficient capacity to hold the oil from several transformers, and also drainage piping from the oil drain cocks on the transformers to the pit

The rotary converters, motor generators, boosters transformers, and frequency changers are better located on the main floor, and whenever possible, the rotaries and motor-generators should be placed in a line.



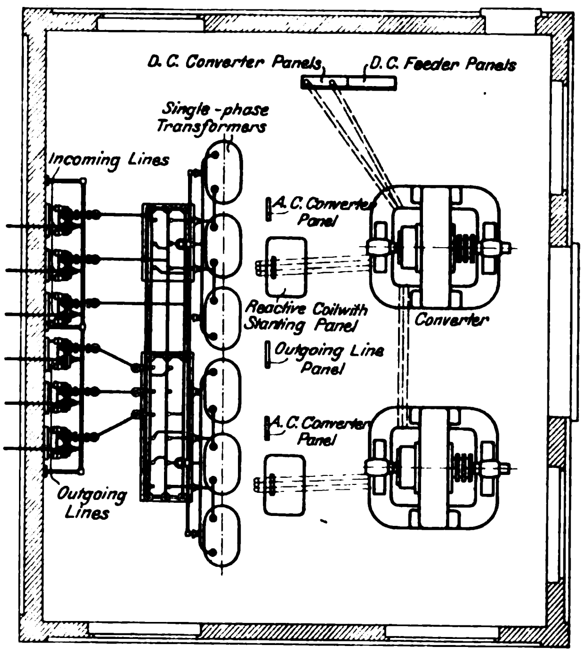


Fig. 220. Plan and Sectional Elevation of Small Substation with Single-phase Oil-insulated Self-cooling Transformers and Hand-operated Oil Switches, 11,000 or 13,200-volt, Overhead High Tension Lines.

The switchboard may be installed, either on the main floor, or a on gallery overlooking the main room.

Roof entrances for the lines are the simplest to install, but such types are exposed to all weather conditions. The protected entrance, usually sheltered by a projection of the roof gives better protection, and affords easy access for inspection and cleaning. The space needed for the low tension portion of the apparatus is usually small, as happens with the housing of high tension switch and bus equipment, if one single transmission supplies the power. Fig. 221 shows the elevation and plan of a 3000 Kw 100,000 volt substation as built by the Southern Power Company.

The substations of the Central Colorado Power Company contain a number of interesting features for the high tension engineer, as they represent advanced practice in the handling of 100,000 volt lines. The Denver substation[1] illustrated in fig. 222 and fig. 223 is for the purpose of stepping down from 100,000 to 13,000 volts, at which energy is transmitted to the Denver Gas & Electric Company's

[1] Electrical World. Vol LV No. 26.

station where it is again transformed for ordinary central station distribution.

The two distinctive features of this substation are, first, the use of outdoor busbars for the 100,000 volt lines, and second, the housing of the transformers, high tension oil switches and lightning arresters in separate buildings. A third feature which may also be mentioned as being somewhat unusual, is the use of a transformer repair house adjoining the transformer house, with special provisions for wheeling transformers out of their regular place in the transformer house, and hoisting their coils for inspection and repair.

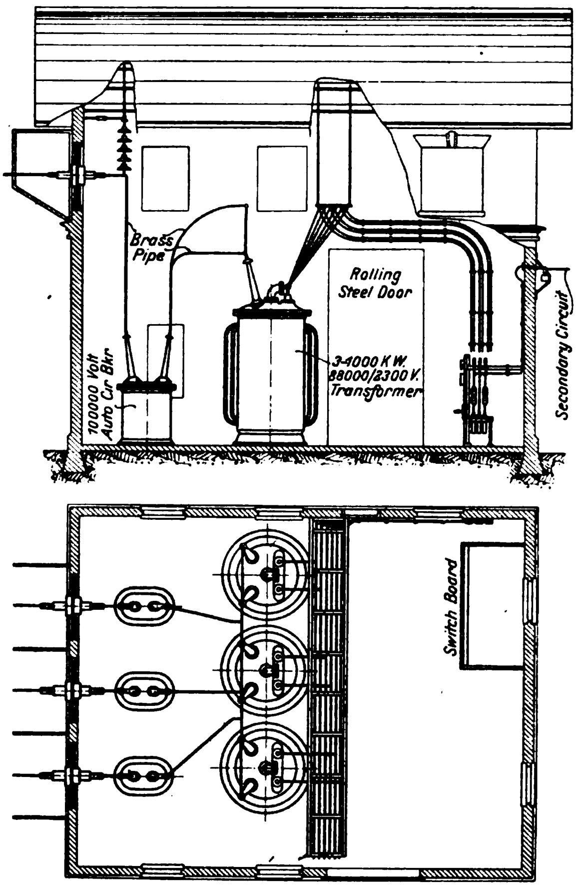


Fig. 221. Elevation and Plan of a 3000-Kw, 100,000-Volt Substation.

There are no 100,000 volt busbars or switching apparatus in the substation building proper. Each of the two incoming 100,000 volt lines has a separate switch house to itself, in which is an oil switch. A diagram of the 100,000 volt connections of the Denver sub-station is given in fig. 224. Each incoming high tension line first passes through disconnecting switches, which are arranged out doors, being supported by and shunted around suspension porcelain insulators of the same type as used on the high tension line. Each insulator consists of four disks 10.25 inches in diameter, arranged in series. From the disconnecting switches each line passes down to the oil switch in the switch house. From the oil switch the lines go up to a second set of disconnecting switches and then to the busbars. These bars are divided in the middle by disconnecting switches. From the busbars, leads pass through disconnecting switches to the transformers. At present there is only one set of transformers in use. The connections of the future transformer set are indicated by dotted lines. The busbars are suspended below the leads, which pass directly from the switches to the transformers, and thus serve to connect together the incoming lines when the disconnecting switch at the middle of the busbars is closed. It is intended in future to put oil switches between the busbars and the transformers at the point indicated in

the diagram. Until the second bank of transformers is added this is not necessary.

A plan of this substation is given in Fig. 223. The three 100,000 volt wires come through the roof in the roof bushing directly over the transformers, and pass straight down to the transformer terminals (fig. 222). Besides this, the only 100,000 volt wiring necessary in the substation is the connection from one end of the transformer bank to the other.

These bushings are from 9 to 11 ft. long according to local requirements, and about 5 ft. extend above the roof. The conductor is held in the middle of a long fiber tube. The space around the conductor and the tube is filled with a thick insulating compound. The diameter of the tube is about 8 inches. The top of the tube is protected with porcelain disk insulators, five in number, and by a piece of sewer tile, which extends from the lowest insulator to the roof. The entrance bushings were at first put into a hole cut through the concrete roof. A number of failures convinced the company that the contact of the bushings with the concrete roof and its reinforcements put an undue strain on the bushing, and the roof was therefore cut away for an area of about two feet square and the space thus made was roofed in with lumber which had been treated with paraffine. No further trouble was experienced. The transformer leads inside the substation consist of brass tubing of the same size as 1 inch extra heavy iron pipe.

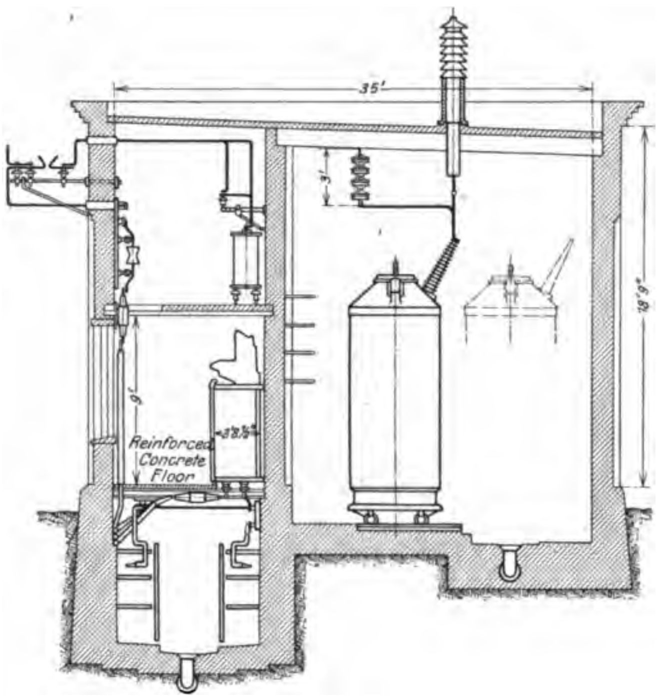


Fig. 222. Denver Substation Section.

To avoid the difficulty, complication and expense of insulating series transformers for the overload relay circuits in connection with the 100,000 volt oil switches in the substations, the coils which actuate the relays are placed directly in the 100,000 volt line as they lead into the switches. The armatures of these relays are connected mechanically with the low tension make-and-break contact in the Denver substation, and in the Dillon substation and others; where switch operation is by hand they operate a mechanical trip instead of closing a direct current control circuit. This mèchanical connection with the relay contacts or with the mecanical trip is made through a long wooden rod boiled in paraffine. It is found that a length of from 10 to 12 ft. is sufficient for insulating purposes.

It will be seen that the transformer room is separated from everything else in the substation. It is isolated by a 13 in. wall from the switching

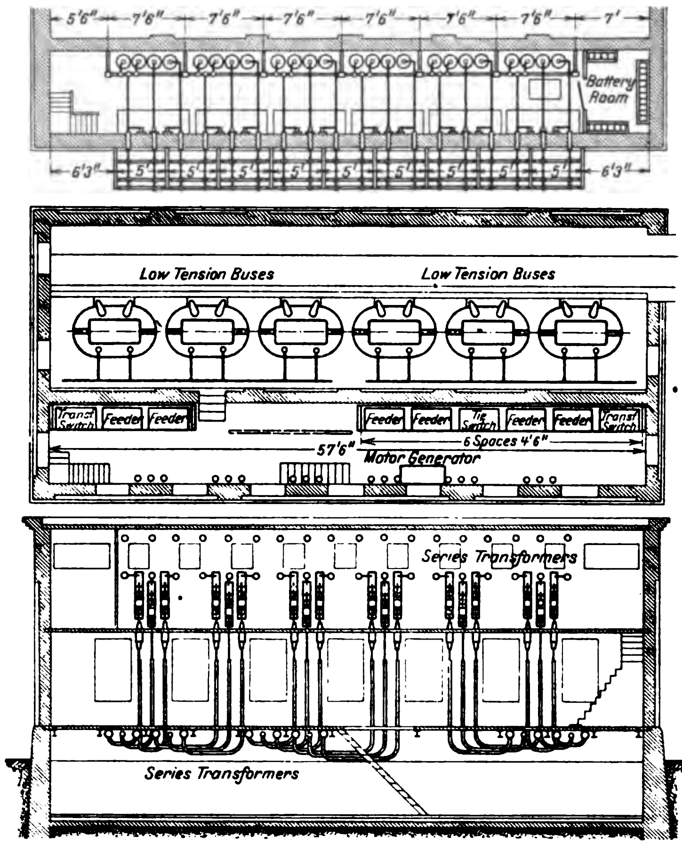


Fig. 223. Denver Substation Plans.

apparatus and 13,000 volt busbars which occupy the other part of the station. Each transformer may be rolled onto a small truck, so that it can be wheeled out into the transformer repair shop. The 13,000 volt busbars are in the basement of the operating department of the substation, above this basement is a room for the 13,000 volt oil switches, and the control switchboard. The third floor is a gallery, which is occupied by lightning arresters and miscellaneous apparatus.

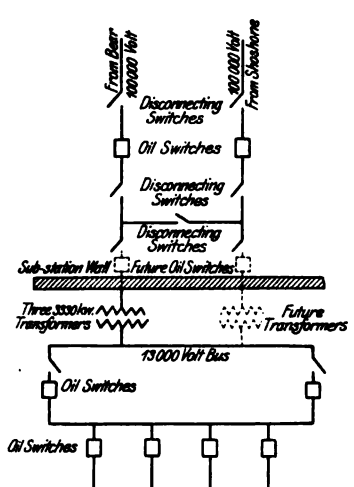
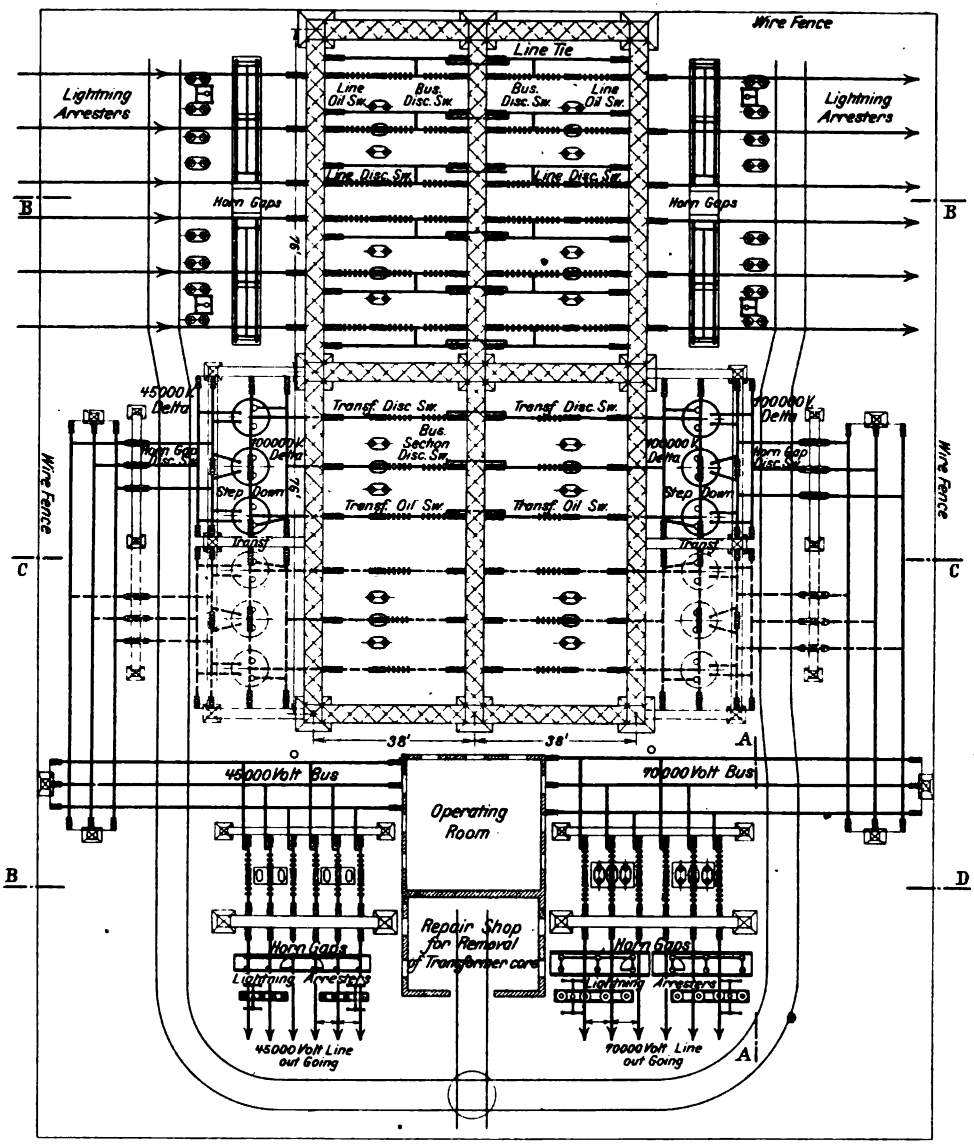
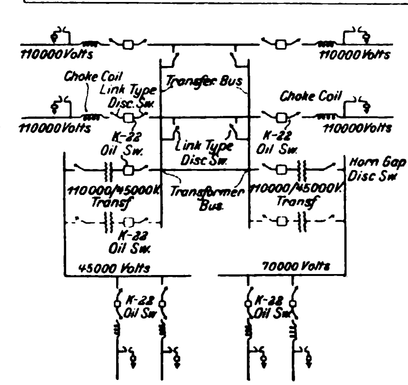


Fig. 224. Connections of 100,000 volt Busbars, Denver Substation.

Aluminium cell arresters are used both on the 13,000 volt and the 100,000 volt lines entering the substation. On the 13,000 volt line, horn gaps are placed just outside the substation as shown in fig. 222.

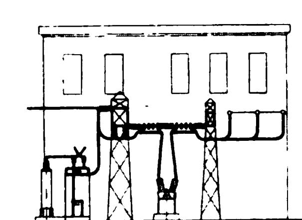
A small storage battery, consisting of 5 cells, made by the Electric Storage Battery Company, is installed for operating the switch motors at the substation. To





Wiring Diagram.

Note—Ground covered = 194 ft. × 250 ft.



Elevation.

Fig. 225. Plan of Outdoor Substation for 110,000/45,000 Volt Circuits.

charge the battery, a 10 HP induction motor, connected to a 6.5 kw. generator is used.

In the basement, below the operating room, the busbar compartments are made of asbestos composition, supported by thin steel sections.

The substation building is of plain brick, with concrete reinforced slab roof. The eye beams supporting these slabs are fireproofed by concrete. The substation and switch houses are as fireproof as possible, having absolutely no wood in their construction except the doors, the building being of brick with concrete roof, metal window frames, sash, etc.

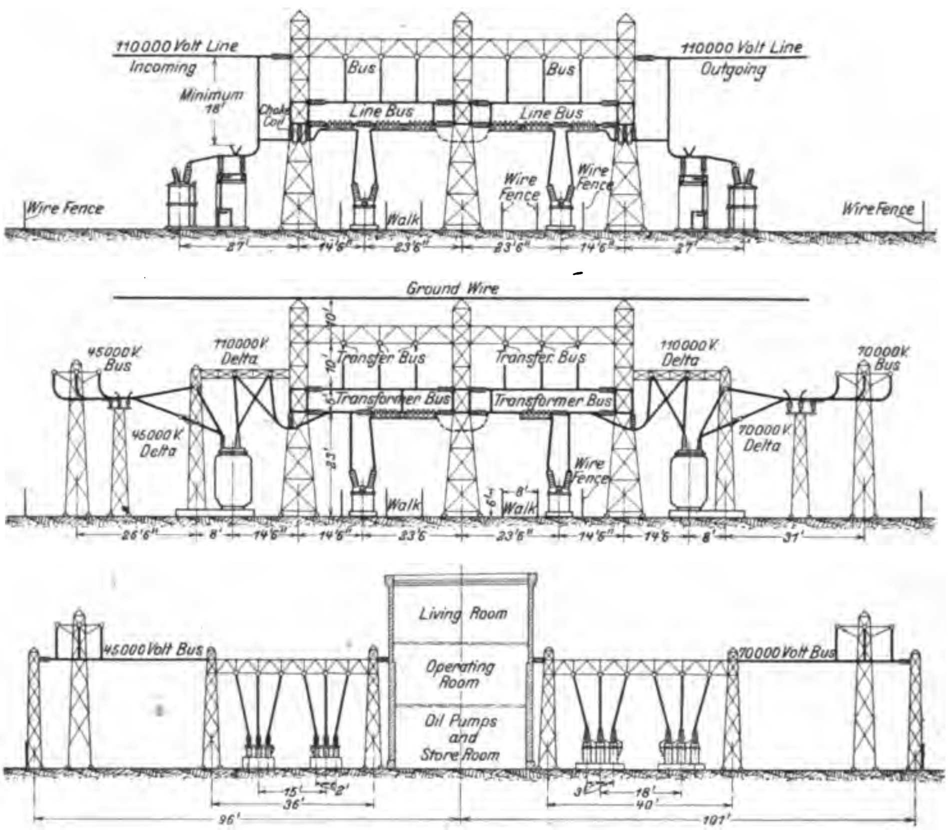
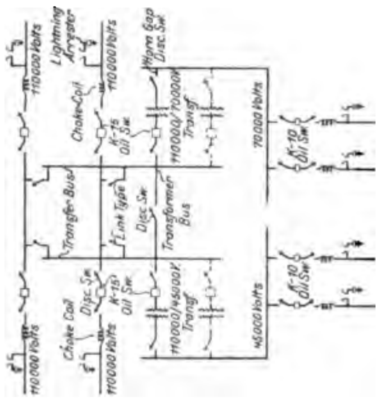


Fig. 226. Sections of Outdoor Substation for 110,000/45,000 Volt Circuits. Sections taken in direction of arrows shown in Fig. 225.

It is interesting to note that the total cost of all buildings, etc. connected with the substation — that is, all cost except electrical apparatus — did not exceed $ 10,000. As the substation has a rating of 15,000 kw., this of course, figures out about $ 0.66 per kw. which compares very favorably with the costs of other substations.

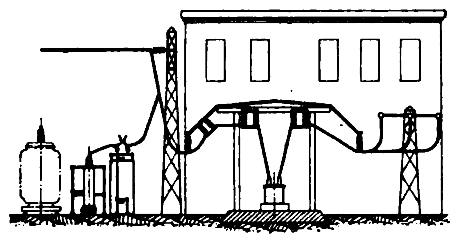
The high tension conductors in a substation should preferably be of tubing with sleeves inside and outside at joints, and supported by either post or suspension insulators, and the distance between phases should be at least 6 ft. with 33 to 36 inches to ground for a voltage of 100,000 volts.

Open wiring is preferable for high tension connections and must be kept as far above the main flooring as possible. Steel columns and frame-



Wiring Diagram.

Note—Ground covered = 230 ft. × 230 ft.



Elevation.

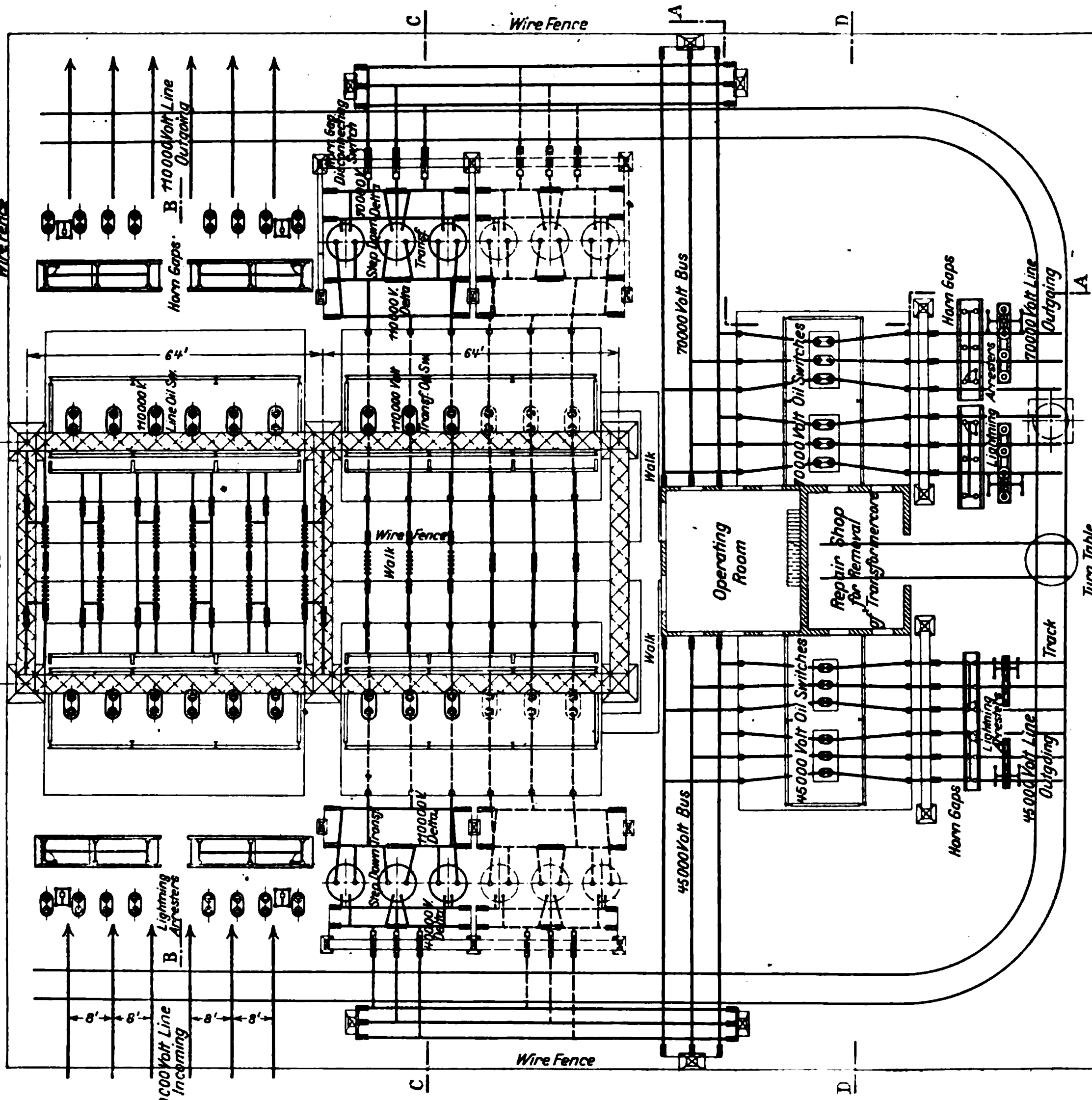
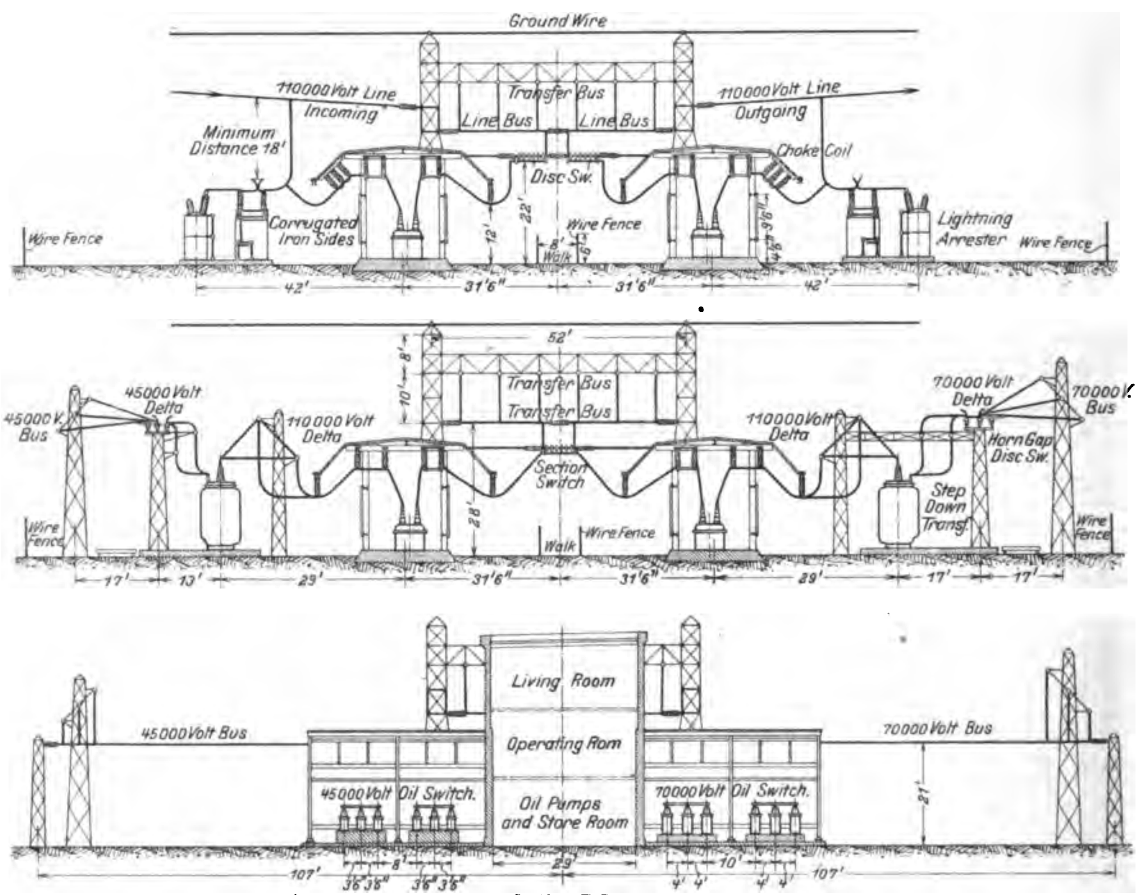


Fig. 227. Plan of Semi-Outdoor Substation, for 110,000/45,000 Volt Circuits.

work in general should be grounded, as the static charges on them may become dangerous. Of late, outdoor and semi-outdoor high tension substations have come into use, the first one being so planned that all of the high tension wiring and apparatus is out of doors, and is merely controlled from a board centrally located in a part of the building.

In the semi-outdoor station, the high tension oil switches and disconnecting switches are protected by means of a light shelter house, the remainder of the high tension wiring and apparatus being out of doors.



Section D-D.

Fig. 228. Sections of Semi-Outdoor Substation, for 110,000/45,000 Volt Circuits. Sections taken in Direction of Arrows shown in Fig. 227.

Table LIV has been developed by C. M. Rhoades[1] to show the cost of the three types of stations, including the indoor type.

The figures throughout are based on heavy capacity switches on the transmission lines with medium capacity switches for transformers and outgoing lines. The two banks of transformers are of 4500 kw. each, with space for two future banks of the same capacity. In the case of the oil-switches, the figures are estimated to include a grade of oil suitable for

[1] General Electric Review, Vol. XV, No. 10.

a climate such as that of the Southern States. If the temperatures are below 32° Fahr., the comparison should be based upon the employment of a higher grade of oil, which would increase the price of the switches approximately 10 per cent. The cost of the building for the indoor arrangement is based upon a building of sheet metal, or metal and cement plaster.

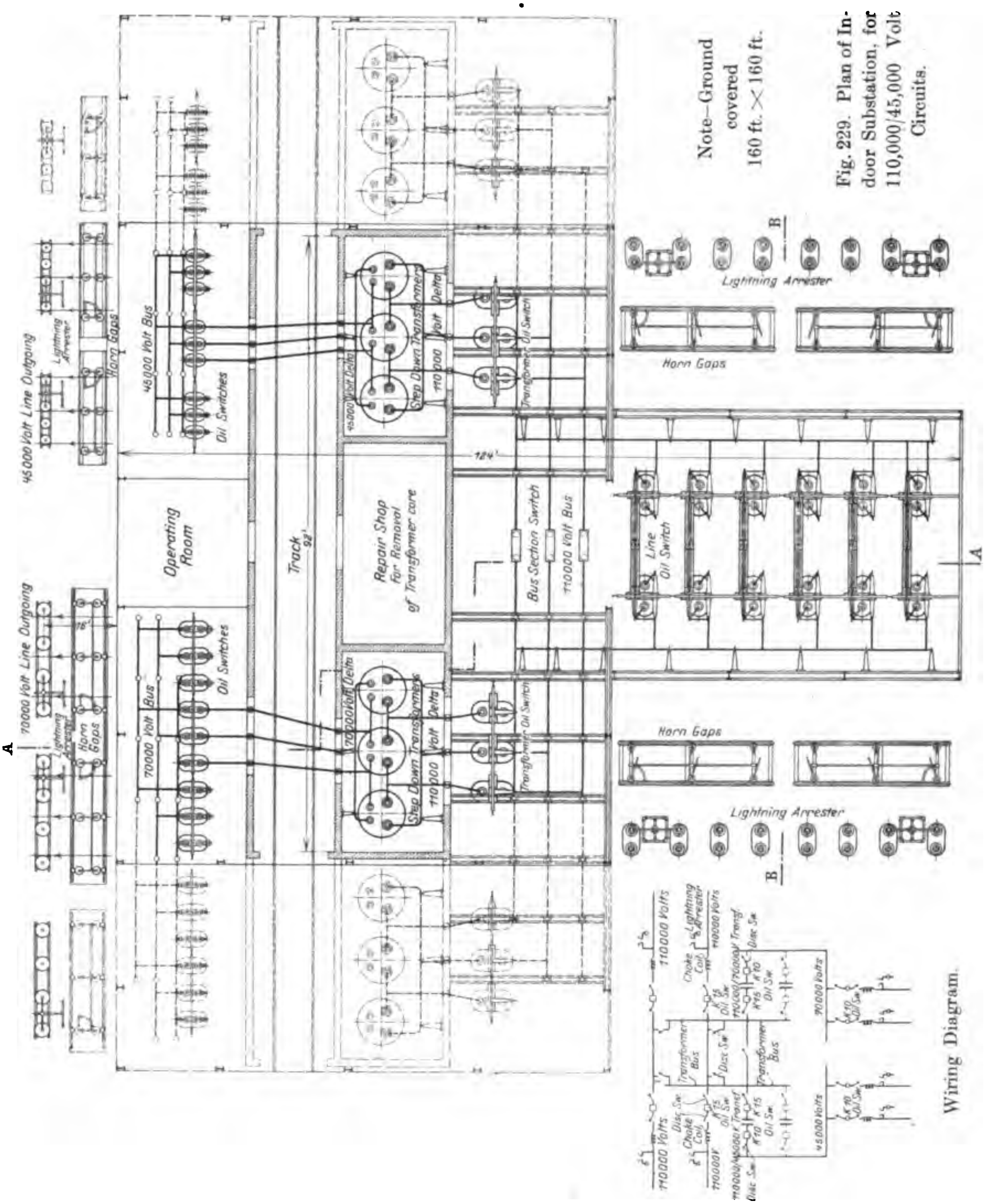


Fig. 229. Plan of Indoor Substation, for 110,000/45,000 Volt Circuits.

to avoid prohibitive expense. If built of brick or concrete the comparison of costs would be quite different.

As shown in the table the cost is in favor of the outdoor and semi-outdoor stations by a very small percentage of the total cost. When operating conditions are considered, the semi-outdoor arrangement has distinct advantages over the outdoor in convenience of operation of disconnecting switches in bad weather, when generally there will be most

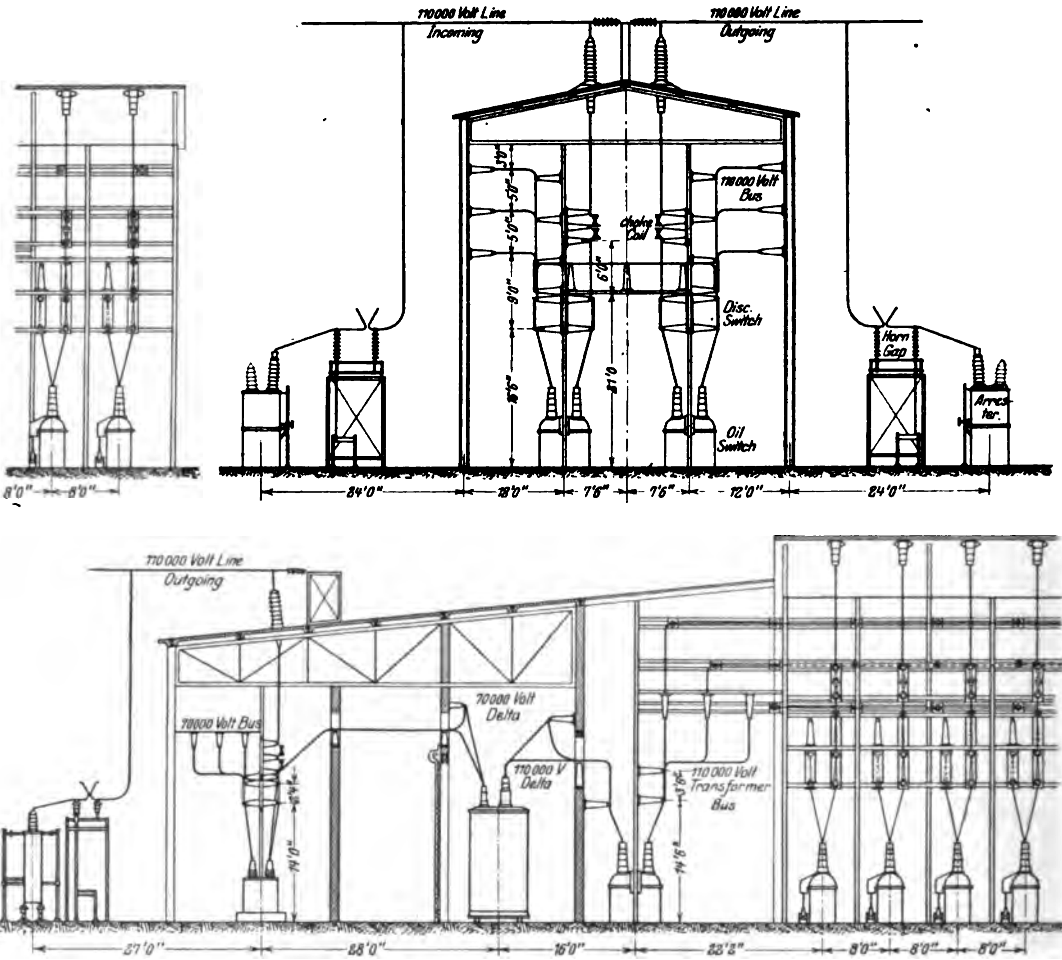


Fig. 230. Section of Indoor Substation for 110,000/45,000 Volt Circuits. Sections taken in Direction of Arrows shown in Fig. 229.

frequent occasion to operate them. The advantage of being able to open and work on the oil switches in place is obvious.

Figs. 225 and 226 show the plan and elevation of an outdoor substation for 111,000/45,000 volt circuits, while figs. 227 and 228 show the plan and elevation of a semi-outdoor substation of the same capacity.

Figs. 229 and 230 show the plan and elevation of an indoor substation of the same capacity.

Table LIV. *Comparative Costs.*

Kind and type of apparatus	Outdoor	Semi-Outdoor	Indoor
Building, foundation, walks, cranes, fences, etc. towers	\$ 20 000.00	\$ 29 000.00	\$ 33 000.00
Electrical equipment, including transformers (110000/70000 volts and 110000/45000 volts), lightning aresters (110000, 70000 and 45000 volts), oil-switches (110000 volts, 70000 kv.-a. rupturing capacity, 70000 volts, 30000 kv.-a. rupturing capacity, and 45000 volts, 30000 kv.-a. rupturing capacity)[1]	\$ 95 414.00	\$ 89 714.00	\$ 89 850.00
Choke-coils, disconnecting switches, bolt-switches, strain and port insulators, wire, tubing and miscellaneous hardware	\$ 8 773.00	\$ 8 910.00	\$ 9 974.00
Labor installation	\$ 8 000.00	\$ 8 300.00	\$ 5 200.00
	\$ 132 187.00	\$ 135 924.00	\$ 138 024.00

Table LV. *Space required.*

Type	Ground dimensions	Ground area
Outdoor	194 ft. by 250 ft.	1.1 acre
Semi-outdoor	230 ft. by 230 ft.	1.2 acre
Indoor	160 ft. by 160 ft.	0.58 acre

129. Foundations for Power House Machinery.

Foundations for hydraulic and electric machinery must be designed with care in order to maintain the machinery in alignment, resist the vibrations occasioned by the machines, and support the full weight of the machinery itself.

The foundations must also be of such proportions so as not to sink. The best method is to employ masonry or concrete foundations, although in some cases, steel structures may be necessary. Transformers are generally placed on insulators, separated from the ground by wooden boards. In the case of turbines and water wheels, it is important that the foundations be of such weight as to absorb the vibrations of the supported machine. For all soils there is a definite safe bearing load, given in table XXXV, page 112.

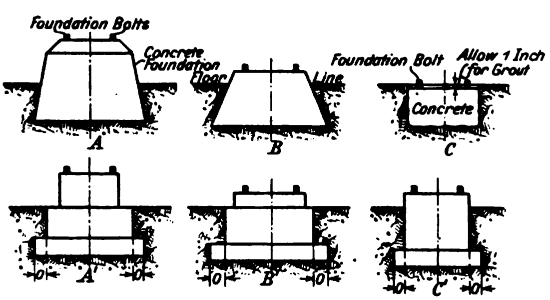


Fig. 231. Designs for Concrete Foundations.

In designing foundations, it is of prime importance to determine the actual weight of the machinery to be supported, together with the weight of the foundation itself; then to provide a sufficient number of square

[1] The rupturing capacity of the 70000 volt and 45000 volt oil-switches included in the figures for semi-outdoor and indoor substations is actually 40000 kv.-a.

feet of area of base to bring the values shown in Table XXXV within a reasonable factor of safety. Lateral displacement may only occur in cases of belt connections, which are little used in hydroelectric plants.

John Young states that foundations should weigh $2\frac{1}{2}$ to 4 times as much as the machines they carry. The design should be as simple as possible, as exemplified in fig. 231[1], otherwise they necessitate intricate, useless and expensive forms. The area of the foundation at the top is determined by the base of the machine and the foundation bolts.

130. Wall Outlets.

Various methods have been used in the passage of high tension lines through walls. Glass and porcelain tubes have been employed, but these are so fragile that they have not been generally adopted. The best solution would consist in leaving clear space around the high tension lines but this is not practicable on account of climatic conditions.

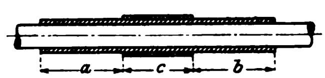


Fig. 232.

Fig. 232, represents the Pertinax type of wall bushing, the outside diameter being 80 mm and the inside diameter 50 mm. The exterior layer *c* is 50 mm in length. The inner layer has a length of 50 mm $+ a + b$. Tests made with this class of insulator gave the results shown in table LVI.

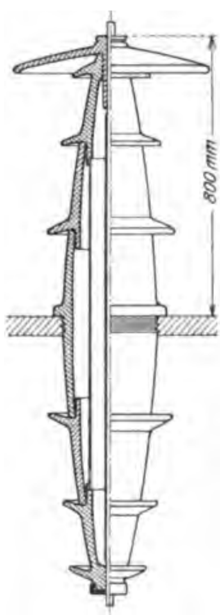


Fig. 233.
A.E.G. 125,000 volts,
Entrance Insulator.

Table LVI.

Length of $a = b$	Breakdown voltage	Length of $a = b$	Breakdown voltage
55 cm.	90 000 volts	20 cm.	70 000 volts
40 „	88 000 „	15 „	63 000 „
30 „	80 000 „	10 „	52 000 „

Fig. 233 represents an A. E. G. insulator, based on the principle of the condenser effect of a bushing. The insulator has a parabolic form, is hollow and filled with a material of low dielectric flux constant. The density of the radial magnetic field diminishes from the axis to the periphery; the dielectric strength and electrostatic rigidity of the materials decrease in the same direction.

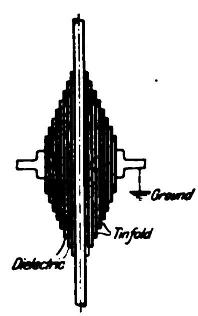


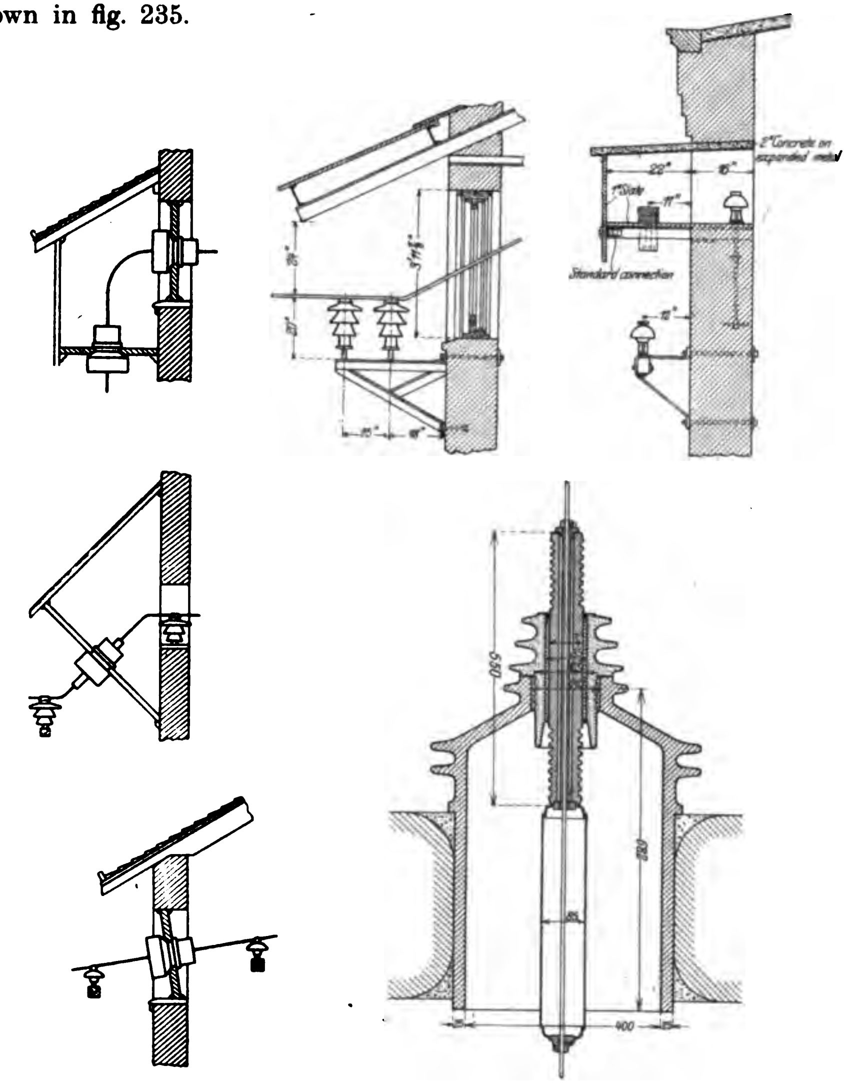
Fig. 234.
Nagel Condenser.

R. Nagel has solved the insulation problem by the use of condensers of nearly equal capacity (fig. 234). His bushing is composed of alternate layers of dielectric and tinfold, with decreasing length to compensate for the increase in diameter so as to keep the area of the condenser approximately constant. This system has given difficulties at the beginning of

[1]) Bruce H. Page, Electrical World, Nov. 3rd 1910.

its use, on account of brush discharge and corona effect, but these difficulties were overcome by the use of a metallic bell attached to the end and metallic rings to each of the first few condenser coatings, thus distributing the dielectric flux.

A number of different types of wall outlets, or line entrances are shown in fig. 235.



Wall Outlet, 50,000-volt Transmission System, Uppenborn, Germany.

Fig. 235. Typical Wall Outlets.

131. Equipment of Standard Switchboard Panels of Medium Capacity.[1]

The following is a brief outline of the instruments usually required for different panels.

Generator panel. The standard equipment of a three phase generator panel is as follows for a tension of 2300 volts (figs. 236 and 237).

[1] General Electric Company's Practice.

Three ammeters,
One polyphase indicating wattmeter,
One voltmeter,
One field ammeter,
One S. P. S. T. field switch with discharge clip,
One hand wheel and chain mechanism for field rheostat,
One four-point synchronizing receptacle,
One eight-point potential receptacle and four-point plug,
One T. P. S. T. non-automatic oil switch,
Two current transformers,
Two potential transformers.

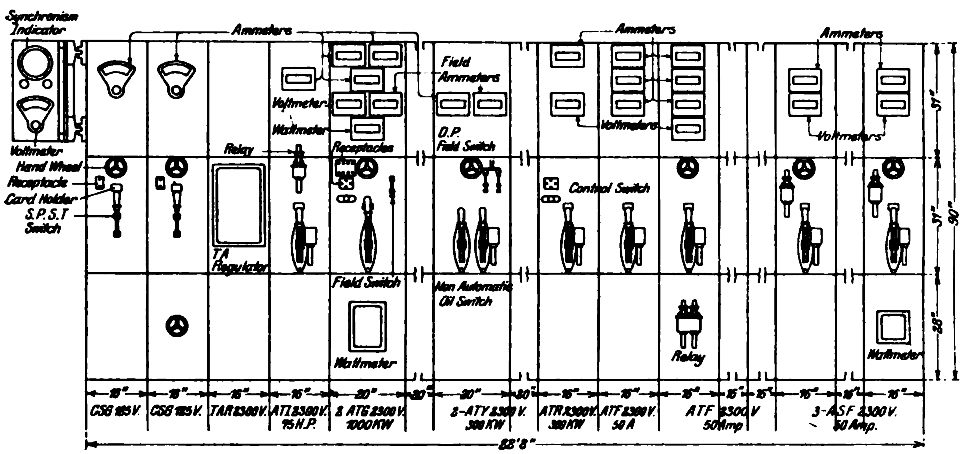
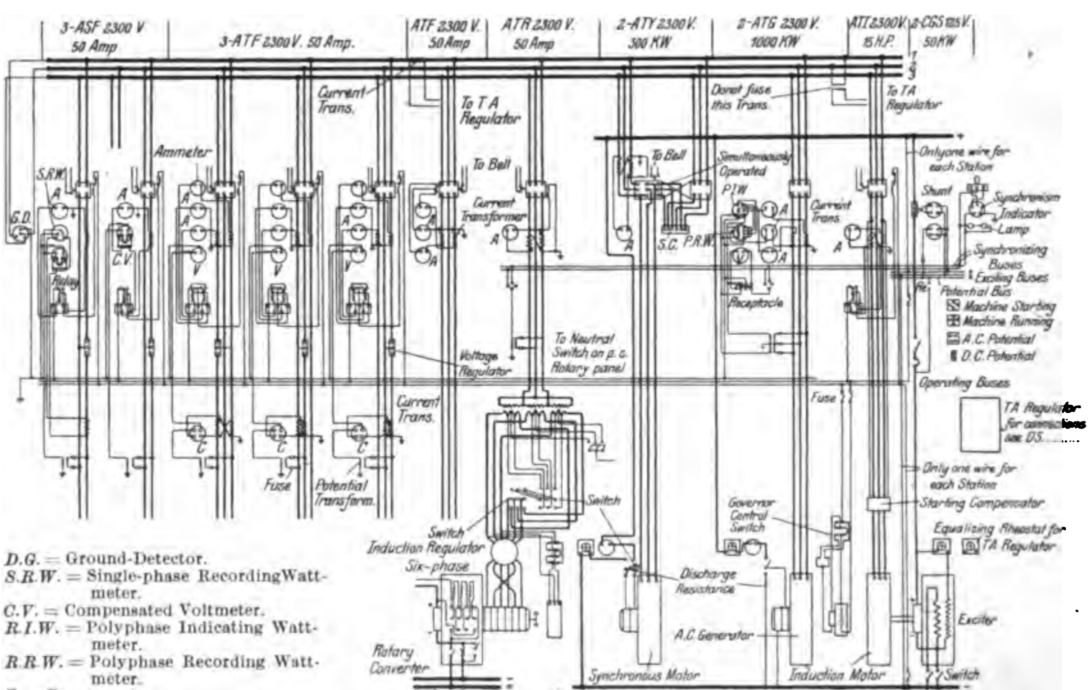


Fig. 236. Standard Switchboard Panels. G. E. Co.



D.G. = Ground-Detector.
S.R.W. = Single-phase Recording Wattmeter.
C.V. = Compensated Voltmeter.
R.I.W. = Polyphase Indicating Wattmeter.
R.R.W. = Polyphase Recording Wattmeter.
C. = Compensator.
S.C. = Starting Compensator.

Fig. 237. Diagram of Connections of 2300 volt Switchboard.

Exciter panel. The equipment recommended for exciter panels is as follows:

One ammeter,
One field rheostat hand-wheel,
One single pole single throw switch,
One two point potential receptacle,
Negative and equalizer switches should be mounted on or near the machines,
A fuse on base, back of panel, may be added,
One voltmeter.

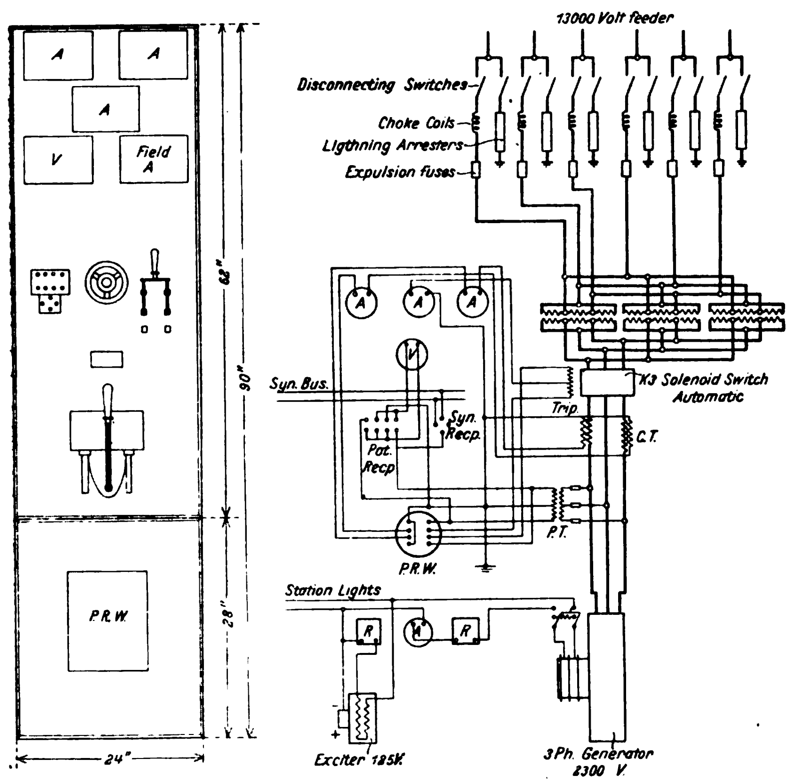


Fig. 238. Standard 2300 Volt, 3-Phase Generator Wiring Diagram with Step-Up Transformers to 13,000 Volts.

Induction motor panel.

One ammeter,
One T. P. S. T. automatic oil switch with bell alarm switch,
One inverse time limit overload relay.

Induction motors are sometimes used to drive exciters, and a special panel for this motor is generally placed between the exciter panel and the generator panel. A Tirrill regulator is often installed on such panels.

Feeder panel. Three phase and single phase.

Three ammeters, only one for single phase panel,
One T. P. S. T. automatic oil switch, D. P. S. T. for single phase,
One shunt transformer,
One wattmeter,
One voltmeter.

11,000 volt alternating current switchboard. *Generator panel.*

One ammeter,
One polyphase indicating wattmeter,
One 175 volt voltmeter,
One field ammeter,
One S. P. S. T. field switch with discharge clip,
One four-point synchronizing receptacle,
One hand-wheel for field rheostat,
One D. P. D. T. engine governor control switch (optional),
One set of 3 S. P. non-automatic switches, operated as a triple pole switch,
Two current transformers,
Two potential transformers.

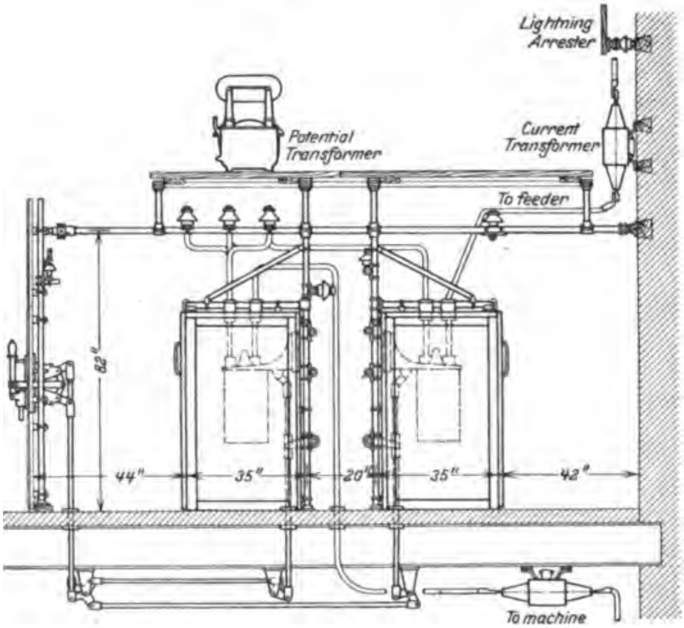


Fig. 239. End View of 11,000 Volt Switchboard.

Exciter panel. The exciter panel has the following equipment:

Two Thomson feeder type ammeters,
One 175 volt Thomson feeder type voltmeter,
Two rheostat hand-wheels,
Two 2-point potential receptacles,
Two S. P. S. T. switches,
One equalizing rheostat,
Negative and equalizer switches should be mounted on or near the machines.
Fuses on base back of panel may be added.

Voltage regulator panel.

One voltage regulator.
One potential transformer.
One current transformer, if compounding is desired.

Induction motor panel.

One ammeter.

One set of three S. P. automatic oil switches, operated as a T. P. switch, with bell alarm switch.

One time limit overload relay.

Two current transformers.

Feeder panels.

Three ammeters.

One set of 3 S. P. automatic oil switches, operated as a triple pole switch, with bell alarm switch.

One time limit overload relay.

Two current transformers.

One synchronism indicator.

There should be provided with each line a set of lightning arresters, with double blade disconnecting switches, one blade being used to disconnect the lightning arrester, and the other to disconnect the line.

Fig. 238 shows the standard 2300 volt three phase generator wiring diagram with step up transformers to 13,000 volts.

Fig. 236 shows the respective location of the various apparatus on the switchboard. The instruments are placed so as to be easily read, and the switches, relays, watt-hour meters are located on the lower part of the board. Fig. 239 shows the end view of an 11,000 volt switchboard.

132. Parallel Operation of Alternators.

When two alternating current generators are connected in parallel but driven by different prime movers with no mechanical connections, the load will be divided proportionally to the power supplied by each prime mover.

If one of the prime movers suddenly decreases its speed, its corresponding generator will drop behind in phase with respect to the other generator unit, and this supplies the power to pull it again in synchronism. In this effort, it gives an impulse to the lagging machine which causes it to swing past normal position, and oscillations are originated until at last they die out and both machines get in step again.

However, if the amplitude of the swinging motion is not damped, i. e. if it remains oscillative, or increases, then the machines are said to be hunting.

The periodicity of the oscillation is given by the expression

$$t = 2\pi\sqrt{\frac{M}{C}}$$

where t = periodic time of hunting

M = mass of revolving parts

C = tangential effort per unit displacement.

The oscillation are overcome by the use of dampers or magnetic friction devices.

Circulating currents are often present in the parallel operation of alternators; they may be due to either of these causes:

a) to a difference in excitation
b) to the phase displacement between voltages
c) to a difference of wave form, producing a higher frequency current.

Therefore, for good operation, generators running in parallel must have the same voltage, the same frequency and the same phase.

In practice, conditions of synchronism are usually indicated by incandescent lamps or by a synchroscope.

133. Railway Converter Substations.

The general equipment of railway sub-stations consists usually of the following apparatus:

a) A high tension switchboard panel for the incoming line, which in most cases carries alternating current. Such a panel is equipped with high tension circuit breakers and high tension switches, connecting the feeders to the static transformers, and low tension switches closing the transformer secondaries on the collector rings. One ammeter is located on the incoming line panel; the switches may be hand- or electrically operated. The overload actuating devices have the form of tripping coils combined with the handle, if the switch is hand operated, whereas overload relays are used for electrically operated switches.

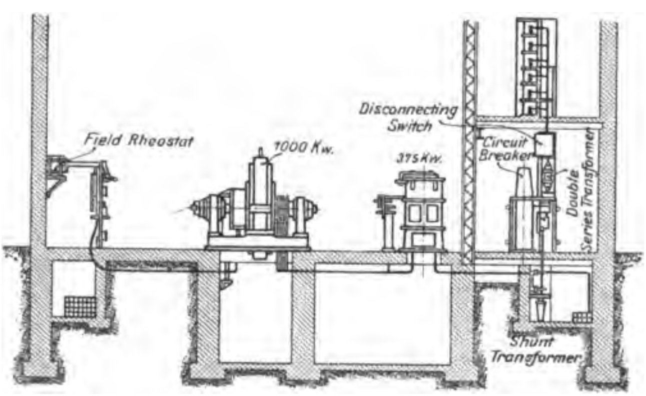


Fig. 240. Typical Rotary Converter Substation. Boston Elevated Ry Co. and Brooklyn Rapid Transit (38th Street).

b) Static transformers, either single-phase or three-phase, which may also be of the air-blast or oil-cooled types. If the transformers are for single-phase current, three of them are necessary for either Y or delta connection with each converter.

c) Rotary converters, intended to transform three-phase alternating current to 600 volts direct current, the supply frequency being either 25 or 60 cycles.

d) Frequency converters, if the primary current is alternating, and of frequencies 25 or 60 cycles, and if a frequency of 15 or 18 cycles for the feeders is necessary.

e) A low tension switchboard with the usual controlling and measuring apparatus. The disposition of this switchboard varies with the method adopted for starting the converter.

The primary tension may be of 11,000; 13,200; 19,100; 33,000, or 66,000 volts. The transformers are generally delta connected for 11,000 13,200 and 19,100 volts, whereas for 33,000 and 66,000 volts, the transformers are preferably Y connected and the neutral is grounded. All of the apparatus used for substation work is fully expounded in other paragraphs, the general outlines of a typical railway converter substation being represented in fig. 240 which is the general layout of a station of the Boston Elevated Ry. Co., and the Thirty-Eighth Street station of the Brooklyn Rapid Transit. The characteristic features are: entrance of high tension cables into station, wiring to oil switches through disconnecting switches and oil switches. All of the high tension circuits are provided with electrically operated circuit breakers.

134. Frequency Changers.

The frequency changer consists of a synchronous motor-generator set, and is used to convert power from one frequency to another.

Fundamentally, it is an alternating current motor mechanically connected to an alternating current generator, bearing to each other the following relation

$$\frac{f}{f'} = \frac{n}{n'}$$

where f and n are the frequency and number of poles of one machine; f' and n' the same for the other machine.

Another type of converter is the induction frequency converter. It consists of a synchronous motor driving the rotor of an induction motor backward. The stator of the induction motor is connected to the supply circuit. Suppose a 25-cycle supply line. At standstill, the frequency of the electro-motive force generated in the rotor windings is 25-cycles. Driving the rotor backwards at synchronous speed develops a frequency of 50-cycles in the second circuit. A frequency of 60-cycles will be obtained if the roter is driven backward at $14\,^0/_0$ of synchronous speed.

The induction frequency convertor has a poor voltage regulation.

135. Motor Generators and Rotary Converters.

There are two types of rotary machines in use for converting alternating current into direct current: motor-generators and rotary converters.

A motor-generator consists of a synchronous or induction motor, mounted on the same shaft with a direct current generator, both machines having therefore separate fields and armatures. The synchronous motor type is used where constant speed is of prime importance and where line compounding or compensation for low power factor is desired, a condition easily obtained by adjustment of the field.

If an induction motor drives the direct current generator, better regulation is obtained, but the efficiency of the combination is only $85\,^0/_0$ at rated load, which is less than that of a unit containing a synchronous machine.

The rotary converter is a machine in which both the alternating and direct currents are in the same armature conductors and there is a definite relation between their e. m. fs.

The chief characteristic of this machine consists in the fact that the closed-circuit armature revolves in a field excited by direct current and is connected on one side to a segmental commutator and on the other to collector rings.

If E is the electro-motive force of the direct current, it can be demonstrated that the alternating e. m. f. E_1 bears the following relation to E in the case of a two pole machine:

$$E_1 = \frac{E}{\sqrt{2}}; \text{ where } E_1 \text{ is the effective value.}$$

In other words, the alternating e. m. f. produced between two collector rings connected with two opposite points of a commutator is equal to the

$$\text{D. C. volts} \times \frac{1}{\sqrt{2}}$$

at a frequency equal to the frequency of rotation.

In the above, it has been assumed that the commutator is connected to a pair of collector rings (single phase arrangement). If four equidistant points are connected to four collector rings, the e. m. f. per phase will be as before $E_1 = \frac{E}{\sqrt{2}}$.

If three equidistant points of the commutator are connected to three collector rings, a three phase converter is obtained.

For three-phase Y-connection

$$E_1 = \frac{E}{2\sqrt{2}}$$

and for delta-connection

$$E_1 = \frac{\sqrt{3}\,E}{2\sqrt{2}}.$$

If the rotary converter is run as a motor from the D. C. side and alternating current drawn from the slip rings, the machine is called an inverted rotary.

The efficiency of the rotary converter, taking into account, the transformer efficiency is about 90 %.

A rotary converter may be started in three ways:

a) Starting from the D. C. side, which is preferable if accumulators are used, or if the primary distribution carries a lighting load.
b) Starting from the A. C. side by means of an induction motor having its armature mounted at the end of the converter shaft, and having two poles less than the synchronous converter.
c) Starting from the A. C. side with current obtained from transformer-taps. This method is used most frequently, and in general, starting from the A. C. side is more advantageous, as it requires less time to throw the converter into service.

The method of starting with an auxiliary induction motor will fail if the poles are laminated and if no damping grids have been provided on the pole faces.

By applying polyphase alternating current to the collector rings, (c), and bringing the rotor up to speed as an induction motor, the starting characteristics are identical with those of starting an ordinary synchronous motor if its field is left without excitation and a reduced voltage is applied to

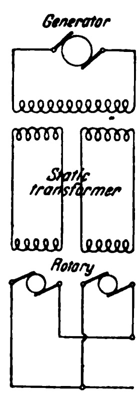


Fig. 241.

the armature. When a shunt to the series coils is used, it must be opened at starting, otherwise excessive heating may be expected.

When the synchronous converter is started by one of the two methods outlined above, it is necessary to use some means for insuring that the converter is running at the proper speed and is in proper time phase with the supply before is closed on the A. C. supply line. With this end in view, incandescent lamps may be used as phase indicators. These lamps may be switched either

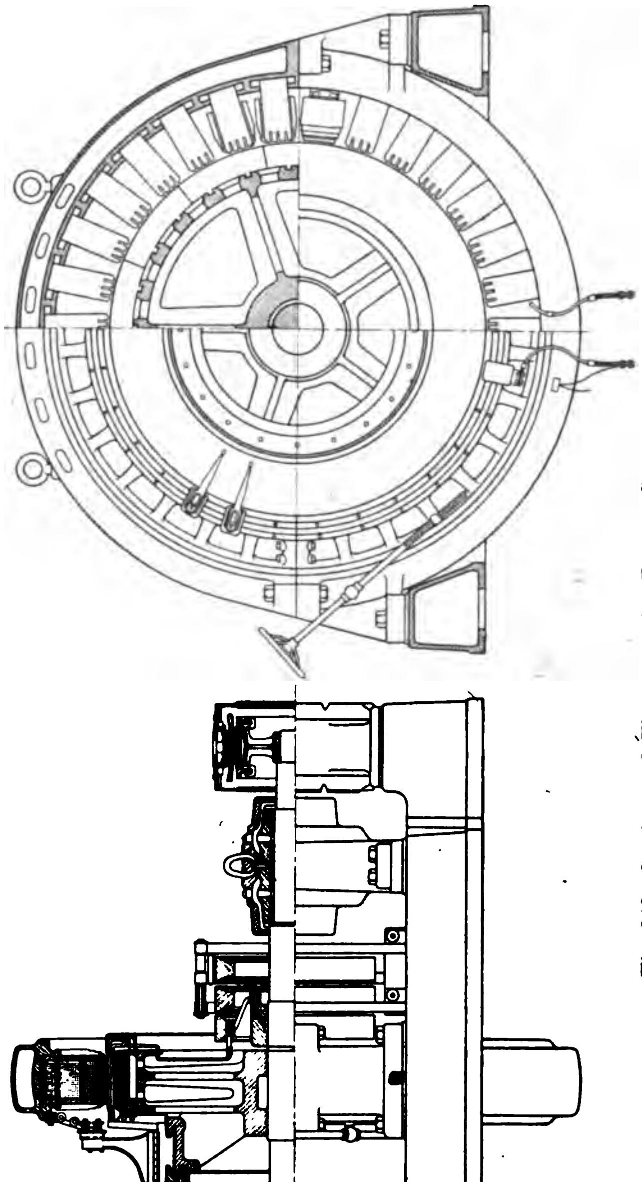


Fig. 242. Section and Elevation of a Rotary Converter.

"dark" or "light", being connected straight across in the first case, and cross-connected in the second.

If the lamps used on a polyphase machine do not flicker in phase with one another, one of the phases on the rotary is reversed and a pair of the supply lines must be reversed. In the case of a polyphase system, the "bright" and "dark" periods should occur in all lamps simultaneously.

If the machine is excited to give a smaller e. m. f. than that of the system when it is running at normal speed, a lagging current will originate which tends to strengthen the motor field, so that the generated e. m. f. is made equal to that of the system. If the field is overexcited, the current will be leading exactly as is the case with synchronous motors.

Therefore, the phase of the current is governed by the excitation.

In the use of rotary transformers, two or more of these machines are sometimes connected in multiple to the secondary of the static transformers[1], and their direct current leads, are connected in multiple to a common bus-bar circuit. With such connections, currents are often formed in the rotaries, that disturb the point of commutation, and it becomes practically impossible to adjust the brushes so they will not spark. Rather than connect across in the above indicated manner, it is better for each rotary to have its own transformer, or at least, its own secondary, on the static transformer as shown in fig. 241.

In order to admit the rocking of brushes over a wide area without sparking, and in this way vary the tension on the direct current side, Miles Walker[2] has divided the main poles into two portions, by cutting away the pole-face for a short portion of the arc. At this part, where the gap is large, a uniform weak field is obtained, proportional to the load, owing to the effect of the series winding.

The brushes themselves are so placed that the e. m. f. generated by this flux assists or opposes the main e. m. f.

[1] Electrical Engineer's Pocket Bock. Horatio Foster.
[2] Electrical Engineering, London.

Chapter VIII. Transmission Lines.

136. Long Distance Transmission of Electrical Energy.

Great attention and considerable study has been given in the United States to the transmission of electrical energy. Recent progress in electrical machinery and apparatus, together with the fertile resourcefulness of the American people have led to many novel and conclusive experiments. Tensions reaching as high as 150,000 volts, in some instances, have permitted the increase of radius of transmission, thereby extending the area of a given energy market. It is not extraordinary to-day to hear of power being transmitted over distances from 200 to 400 miles, and projects are at present in development for supplying power to distances up to 500 miles.

In developments of large magnitude, the power generated at the plant is distributed at high tension to a system of sub-stations, generally located at some market center, where the current is stepped down to a low voltage by means of the necessary transformers.

Three phase transmission offers the advantage of economy in the size of the conductors, and the elasticity of such a scheme of distribution renders it very suitable for the use of asynchronous motors and rotary converters; this explains why a considerable net work of such lines are appearing in this country. Experience shows that an amount of power not exceeding 250 kilowatts may be transmitted over a distance of 6 to 7 miles at a tension of 3,000 volts; for 500 kilowatts over 15 miles, a tension of 5,000 or 6,000 volts is generally adopted. Over thirty miles, a reasonable amount of power may be transmitted at from 10,000 to 15,000 volts. A radius of transmission from the generating plant extending 80 miles may be reached with a tension of 30,000 to 40,000 volts, for greater distances, a system using a working voltage of 60,000 or more must be considered.

The problem of electrical transmission at these high voltages, however, presents some complications; due allowance must be made for the distributed capacity, self-induction and leakage of the line, the latter being apt to appear either as a leakage at the insulators or as a corona effect.

Corona, when begun, increases very rapidly. Its origin is due to the breaking down of the air around the conductor, it becomes conducting and luminous. For voltages above 100,000 volts, it is indispensable to investigate and provide against corona losses.

At Niagara Falls there are five power stations with a maximum output of 300,000 HP, supplying power at a distance of 150 miles at a working voltage of 90,000 to 133,000 volts. For the transmission of power from Mt. Hamilton (California) to Oakland and San Jose, the distance being 150 miles and 200 miles respectively, a pressure of 40,000 volts is in use, until the load on the station is sufficiently large to warrant a tension of 60,000 volts being employed.

In Mexico, where the development of water power has been exploited to a great extent in latter years, a 60,000 volt transmission line is already in existence, as in the Necaxa plant. On the Pacific Coast, large plants have sprung into existence. The station on the Feather River is operated at 100,000 volts over a line of 154 miles.

For the supply of the San Francisco market by the Pacific Gas and Electrical Company, eleven water power plants are connected to the system of distribution; these are coupled in parallel on a single net work covering in all an area of 30,000 sq. miles and extending about 250 miles on the Pacific Coast. The total length of this 60 000 volt overhead line is about 1000 miles.

The Central Colorado's plant operates at a voltage of 100,000 volts and the Grand Rapids plant at 110,000 volts.

Lately, hydroelectric energy from the Des Moines Rapids of the Mississippi River has been developed. The initial installation has a capacity of 150,000 HP, which will be increased to 200,000 KW later, and eventually to 300,000 HP, the latter being the full power of the river at this point. The energy is transmitted over a 110,000 volt, 25 cycle line. It is thanks to the special capacities of engineering and initiative of Mr. H. L. Cooper, the builder of this development, that such a tremendous task has been put into practice. Such an installation will constitute the exponent of american activity and enterprise, at a time when economical interests and industrial developments govern the world and make great nations.

The development of the Victoria Falls (Zambese River) is also under consideration at present, and it is intended to transmit the power to Johannesburg and Pretoria, situated at about 800 miles from the proposed generating station.

It has been noted from experience that above 30,000 volts, not only do the alternators increase in cost, as well as the transformers, but also the difficulties of insulation increase. The distances between wires and between the wires and the ground are determined by considerations of potential difference and the production of a corona effect from the line itself. The corona as has been explained is due to the electric stress produced in the zone around the wires by its electrostatic charge being greater than the air can withstand.

The rarefied air of the higher altitudes is a better conductor than the ordinary atmosphere. A corona is more easily formed at higher altitudes, and the relation between air pressure and electric strength is given approximately by the following empirical expression:

$$R = R_0 \left[0.2 + \frac{0.8p}{760} \right]$$

where p is the atmospheric pressure in millimeters of mercury. The electric strength is also lessened by an increase in temperature. The question of charging current, with its relation to the frequency, must also receive proper consideration. Low frequencies are to be preferred in general, on account of the reduced effects of inductance and capacity. Frequencies of 25, 50 and 60 cycles offer the best advantages, as the difference between weight and cost of equipment is not very marked. For electric traction, a frequency of 25 cycles per second is in general use. The standard frequencies used in America are 25 and 60 cycles. The latter is used for lighting purposes, and can be easily transformed to frequencies of 25 or 50 cycles per second.

The limit of high tension is imposed by the loss of the line, which bears relation to the diameter of the wires and the distances between them. In high tensions, rupture of the air takes place; discharges attaining several inches in length are produced, accompanied by sound and an ozonizing effect of the air.

Table LVII computed by Messrs. Marchant and Watson, gives the stresses existing on the lines and the factor of safety allowed, for transmission lines of the highest existing voltages:

Table LVII.

Plant	Voltage	Conductor B. and S.	Spacing cm	Altitude Ft.	Stress. Kg./cm	Factor of Safety allowing for Altitude
Great Western Power Co. Island Bar	100,000	000	300	800	24.2	1.52
	150,000	—	—	—	33.0	1.11
Central Colorado Power Co.	100,000	000-6 strand with hemp core	300	6.000	23.2	1.34
Grand Rapids	110,000	0000	300	1.000	19.6	1.87

Valuable data on many of the Western transmission systems are given in table LVIII[1] and table LIX[2] gives the characteristics of American power plants with transmission voltage of 70,000 or more.

The Thury system of transmission by means of high voltages and direct current is receiving serious considerations, as it offers some advantages. Although this system of distribution may receive wide application in the future, it must be noted that the construction of high tension alternating current machinery is more easily done than the equivalent direct current machinery; alternating current system offer simpler distribution, and do not produce electrolytic effects such as occur with the direct current system.

[1] Electrical World-Vol. 59. No 22. Page 1195.
[2] A Study of Three-Phase Systems. C. Fortescue. The Electric Journal, Sept. 1914.

Table LVIII. *Comparative line data on transmission systems.*

Owner of Property	Location of Transmission Line	Transmission Distance, Miles	Kilovolts	Cycles	Line Supports	Insulators	Spans, Feet	Minimum Separation of Wires, Inches	Number and Size of Conductors	Rating of Transformers Equipment Kw.
United States Reclamation Service, Roosevelt Dam	Salt River to Phoenix, Ariz.	75	45	25	Steel poles and steel towers	14-in. pin type	400 standard 800 maximum	48	Six 83,000 cir. mil. copper six-strand hemp core	5,000
Southern California Edison Company	Kern River to Los Angeles	122	75	50	Four-column steel towers	18-in. pin type	700 standard	72	Nine No. 4-0 copper, seven-strand	20,000
Pacific Light & Power Company	Kern River to Los Angeles	127	60	50	Two independent pole lines	Pin-type, four-part	540 standard	72	Six No. 3-0 copper	12,000
Pacific Light & Power Company	Big Creek to Los Angeles	275	110	50	Double steel tower	Suspension type	Under construction			120,000
Great Western Power Company	Big Bend to Oakland, Cal.	154	100	60	Double-line steel towers	Suspension type, five-unit	750 standard 2740 maximum	120	Six No. 3-0 copper, seven-strand	40,000
Sierra & San Francisco Power Company	Stanislaus River to San Francisco	138	104	60	Steel-towers four-column	Suspension type, five-unit	850 standard 1600 maximum	96	Three No. 2-0 copper, six-strand, hemp core	90,000
Portland Railway Light & Power Company	Cazadero to Portland, Ore.	28	60	60	Steel towers	Suspension type, thirteen-unit	500 standard 1800 maximum	72	250,000 cir. mil. cable	11,000
Seattle-Tacoma Power Company	Snoqualmie Falls to Seattle, Tacoma and Everett	32 43 50	60	60	Cedar poles	Four-piece porcelain pin	140 standard 1000 maximum	84	No. 4-0 and No. 2-0 aluminum cable	20,750
Washington Water Power Company	Little Falls to Wallace	100	60	60	Steel towers	Suspension type, four-unit	650 standard 910 maximum	84	270,000 cir. mil. aluminum cable	22,000
Great Falls Power Company	Missouri River to Anaconda and Butte, Mont.	150	102	60	Double steel tower line	Suspension type, ten-unit	600 standard 3034 maximum	124	No. 0 copper, six-strand cable	21,500
Telluride Power Company	Grace, Idaho, to Salt Lake City, Utah	128	44	60	Wooden poles and steel towers	Porcelain and glass on wood pins	300 standard 900 standard	72	No. 6 copper and No. 8 aluminum	12,000
Central Colorado Power Company	Glenwood to Denver	152	100	60	A-type steel towers	Suspension type, four-unit	600 standard 2700 maximum	124	Three No. 0 copper, six-strand, hemp core	15,000
Grand Rapids-Muskegon Power Company	Big Rapids to Grand Rapids, Mich.	50	110	30	Triangular steel towers	Suspension type, five-unit	528 standard	96	Six No. 2, six-strand copper with hemp core	10,000
Commonwealth Power, Railway & Light Company	Au Sable to Flint, Mich.	235	140	60	Triangular steel towers	Suspension type, ten-unit	500 standard	144	Three No. 9 seven-strand copper	10,000
Ontario Hydroelectric Power Commission	Niagara Falls to Toronto, Can.	90	110	25	Double circuit steel towers	Suspension type, eight-unit	550 standard	96	Six No. 4-0 and No. 3-0 aluminum	40,000
Shawinigan Water & Power Company	Shawinigan Falls to Montreal, Can.	85	100	60	Double-circuit steel towers	Suspension type, seven-unit	520 standard 1400 maximum	96	Six 250,000 cir. mil. aluminum cable	40,000
Southern Power Company	Great Falls, S. C., to Durham, N. C.	100	100	60	Double-circuit steel towers	Suspension type, six-unit	600 standard 1600 maximum	100	Six No. 2-0 copper and aluminum cables	24,000
Lauchhammer Aktiengesellschaft (German High-Tension Plant)	Coal Mines at Lauchhammer to German Iron Works	35	110	50	Double-circuit lattice steel poles	Suspension type, five-unit	550 standard 900 maximum	70	83,000 cir. mil. copper cable	20,000
Georgia Power Company	Tallulah Falls to Atlanta, Lindale and Newnan	90 160 170	110	60	Double-circuit steel towers	Suspension type, five-unit and six-unit	550 standard 1200 maximum	108	No. 4-0 and No. 2-0 copper	60,000

Table LIX. *American Power Companies transmitting at 70 000 volts and above.*

Name	Operating voltage	Frequency (cycles)	Capacity of plant (kilowatts)	Beginning of operation	Phases	Step-up Trans. Connections Low tension	Step-up Trans. Connections High tension	Y-grounded direct or through resistance	Step-down Trans. Connections Low tension	Step-down Trans. Connections High tension	Dist. of transmission (miles)
Pacific Light & Power Co	150 000	50	59 500	1913	1	Δ	Y	D.	Δ	Δ	241
Au Sable Elec. Co	140 000	60	19 000	1912	1	Δ	Δ	No	Δ	Δ	245
Southern Sierras Power Co	140 000	60	8 750		1	Δ	Y	R	Δ	Y	239
Utah Power & Light Co	130 000	60	42 000	1914	1	Δ	Δ	No	Δ	Δ	135
Pacific Gas & Electric Co	125 000 to 110 000	60	25 000	1913	1	Δ	Y	D.	Δ	Y	110
Tennessee Power Co	120 000	60	15 000	1914	3	Δ	Δ	No	Δ	Δ	140
Connecticut River Transmission Co	120 000	60	14 400	1914	3	Δ	Y	D.	Δ	Y	60
Inawashiro Hydro-Electric Power Co	115 000	50	42 000	1914	1	Δ	Δ	No	Δ	Δ	144
Grand Rapids Muskegon Power Co	110 000	30	9 000	1906	1	Δ	Δ	No	Δ	Δ	35
Lauchhammer, A. G.	110 000	50	15 000	1911	3	Y	Y	No	Y	Y	35
Ontario-Hydro-Electric Commission	110 000	25	106 800	1910	1	Δ	Y	R	Δ	Y	90
Georgia Ry. & Power Co	110 000	60	50 000	1912	1	Δ	Y	R	Δ	Δ	210
Alabama Interstate Power Co	110 000	60	48 000	1913	1	Δ	Y	R	Δ	Y	150
Mississippi River Power Co	110 000	25	112 500	1913	3	Δ	Y	D.	Δ	Δ	144
Mexican Northern Power Co	110 000	60	26 000	1914	1	Δ	Y	R	Δ	Y	157
Lehigh Coal and Navigation Co	110 000	25	30 000	1914	1	Δ	Y	R	Δ	Δ	24
Ebro Irrigation & Power Co., Ltd.	110 000	50	50 000	1914	1	Δ	Δ	No	Y	Δ	105
Chile Exploration Co	110 000	50	40 000		3	Y	Y	No	Y	Y	86
Sierra-San Francisco Power Co	104 000	60	34 000	1910	1	Δ	Y	D.	Δ	Y	138
Yadkin River Power Co	100 000	60	24 000	1912	3	Δ	Y	D.	Δ	Δ	96
Great Falls Water Power & Townsite Co	102 000	60	21 000	1910	1	Δ	Δ	No	Δ	Δ	150
Central Colorado Power Co	100 000	60	10 000	1909	1	Δ	Δ	No	Δ	Δ	152
Great Western Power Co	100 000	60	50 000	1909	3	Δ	Δ	No	Δ	Δ	154
Southern Power Co	100 000	60	75 000	1909	1	Δ	Y	R	Δ	Δ	210
Shawinigan Water & Power Co	100 000	60	45 000	1911	3	Δ	Y	R	Δ	Δ	87
Los Angeles Aqueduct	100 000	50	22 500	1914	1	Δ	Y	D.	Δ	Y	47
Tata Hydroelectric Co	100 000	50	40 000	1914	I	Δ	Δ	No	Δ	Δ	43
Appalachian Power Co	88 000	60	20 610	1912	3	Δ	Δ	No	Δ	Δ	75
Rio Janeiro Tramway, Light & Power Co	89 000	50	48 800	1913	1	Δ	Δ	No	Δ	Δ	51
Sao Paulo Elec. Co	88 000	60	30 000	1914	1	Δ	Δ	No	Y	Δ	56
Mexican Light & Power Co	85 000	50	58 500	1910	1	Δ	Y	D.	Δ	Y	169
Victoria Falls & Transvaal Power Co	84 000	50	40 667	1913	3	Δ	Y	R	Y	Δ	30
Toronto Power Co	85 000	25	80 000	1914	1	Δ	Δ	No	Δ	Δ	80
Katsura Gawa Denryoku Kabushiki Kaisha	77 000	50	22 400	1912	1	Δ	Y	R	Δ	Y	48
Southern California Edison Co	75 000	50	20 000	1907	1	Δ	Y	D.	Δ	Y	117
Hidro-Electrica Espanola Molinar	70 000	50	22 500	1910	3	Y	Y	No	Y	Y	158
Pennsylvania Water & Power Co	70 000	25	71 500	1910	3	Δ	Y	R	Y	Δ	40

137. Transmission Line Calculations.

With steadily rising voltages and increasing distances of transmission, the problem of energy transmission has become of prime importance. The problem presents itself under two aspects: the electrical and the mechanical considerations. In both cases, methods of analysis exist in profusion, and a large number of formulae are given.

In any development and long distance transmission of electrical energy, the transmission line is usually the largest part of the investment; and it is obvious that the system of distribution requiring least copper, i. e. the three phase system, can be considered with advantage, although the high tension direct current system has in some instances proven practical.

The relative amounts of copper required for various systems and for the same difference of potential between wires is given herewith:

Single phase system	100
Two phase four wire	100
Two phase three wire	146
Three phase three wire	75
Direct current	50

It must be noted that the weight of conductor is inversely proportional to the square of the voltage, therefore the higher the potential the better, at any rate, so far as this is consistent with good engineering. Following is a recapitulation of the relations between the length of line and voltage which have been found practical:

Distance in miles.	Voltage of line.
6—7 miles	3000 volts
15 "	5000 to 6000 volts
30 "	10,000 to 15,000 "
80 "	30,000 to 40,000 "
100 "	60,000 "
100—200 "	60,000 to 150,000 "

Considerable diversity of opinion exists in the choice of tensions and the above must not be considered as standard. Mr. Still has derived the following empirical formula which may be used for preliminary estimates and which agrees generally with modern practice:

$$E = 5.5\sqrt{l + K}$$

in which

$E =$ tension in kilowatts,

$l =$ length of transmission line in miles,

$k = \frac{\text{horsepower transmitted}}{200}$.

The highest voltage that can safely be used in connection with the pin type of insulator is 44,000 volts. The suspension-type insulator is used for higher voltages, and its cost is comparatively moderate. It can be said, in the present state of the art, that the voltage limit of to-

day is due to corona loss from the line conductor, but not from the line insulator.

The cost of long distance transmission depends on the voltage. It decreases (as far as the conductors are concerned) with the square of the tension, but the cost of insulators increases with the voltage. This is also true for the pole-line, because high tension requires greater distance between conductors, longer or higher poles, long cross-arms, heavier construction. As to the spacing of the conductors, a basis for calculations may be taken at one foot for every 10,000 volts between conductors.

The critical length of a transmission line may be found by the formula:

$$f = \frac{450}{l} \text{ or } l = \frac{450}{f}$$

in which

f = natural frequency of the system; and
l = length of the line in hundred miles.

If L_m in milhenrys,
C_m in microfarads,

for concentrated inductive and condensive reactances, the natural frequency is determined by:

$$f = \frac{5050}{\sqrt{L_m C_m}}$$

for distributed inductive and condensive reactances, as is the case in transmission lines:

$$f = \frac{7900}{\sqrt{L_m C_m}}.$$

As has been said, there is a limitation to the increase of voltage on transmission lines due to the breakdown of the dielectric, the limiting voltage being defined by the expression:

$$E_{max} = 110\,(2\,r)^{0.8} \log_{10} \frac{d}{2\,r} \text{ kilovolts}$$

in which

E_{max} = maximum value of the voltage,
r = radius of the conductor in inches, and
d = interaxial distance between parallel wires, also in inches.

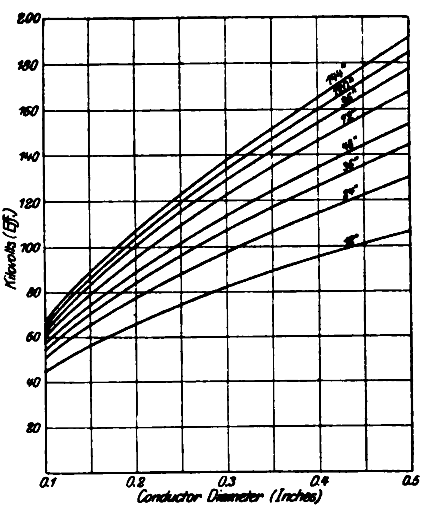


Fig. 243. Critical Voltages between Conductors in Three-Phase Circuits.

The critical voltages, between conductors in three phase circuits are shown by the curves[1] (fig. 243), which were plotted from the above formula. The effective voltages values corresponding to the appearance of corona are given for conductors No. 10 to No. 0000 B. & S. at spacings from 1 to 12 ft.

[1] Erich Hausmann, Electrical World. Nov. 17, 1910.

From the mechanical point of view, the calculation of stresses and sags in wire spans is a very important problem, which could be rendered easier if there existed comprehensive data in relation to the sleet and wind loads; not only should such data cover wind, sleet and temperature independently, but also show simultaneous values in all possible combinations. In assigning values to the maximum stresses to which the line will be subjected, the designer of to-day is merely assuming that it is probable that such values will not be exceeded in practice. The stresses caused by wind velocities and sleet loads are as important as reliable data relating to them is scarce. The best policy involving evidently an error on the side of safety, consists in assuming always the simultaneous existence of the maximum known wind velocity and maximum probable load of sleet with the respective degree of temperature.

Steel towers have become the favorite form of construction on large systems. In the present state of the art, they are considered as cantilevers, not as columns. The external loads are due to dead, ice and wind loads. The weight of the conductors and their coating of sleet, the weight of all inherent material, cross arms, insulators and the tower itself, are loads which it carries as a column. The wind forces act at the top of the towers, where the wires are attached, and if it is placed at a bend, the horizontal component of the tension must be added.

The best type of steel tower is undoubtedly the Semenza type, consisting simply of a pole or tower, that will bend within its elastic limit, thus reducing the strain on the line. In such an arrangement, anchorage towers of greater stiffness are placed at intervals. There are many other considerations besides lightning protection to be kept in mind in designing transmission lines, but these are different in each case, such as facility for effecting repairs, insulator cost, and the proximity of a telephone line.

In a continuous current circuit, when the current is constant, the power supplied to the circuit is IE. In an alternating current circuit, considering that the current and e. m. f. are not in phase, the above condition is not usually realized.

The alternating current is a harmonic function of time, represented by

$$i = I_0 \sin \omega T$$

where

i = instantaneous value of current,

I_0 = maximum „ „ „ ,

$\omega = 2\pi f$, where f is the frequency of the current,

T = period of one alternation $= \frac{1}{f}$

If the current i flows in a circuit containing resistance, inductance and capacity in series, the impressed e. m. f. must equal at every instant the sum of the reactions, i. e:

$$e = \sum \text{ of reactions}$$

$$= iR + \frac{di}{dt} L + \frac{Q}{C}$$

where Q is the instantaneous charge of the condenser and C its capacity. The solution of this equation gives as the value of virtual current:

$$I = \frac{E}{\sqrt{R^2 + \left(2\pi f L - \frac{1}{2\pi f C}\right)^2}} = \frac{E}{\sqrt{R^2 + X^2}} = \frac{E}{Z}$$

The denominator of this fraction is called the impedance of the circuit, its value is expressed in ohms and it is represented by the symbol Z.

The expression $\left(2\pi f L - \frac{1}{2\pi f C}\right)$ is the reactance of the circuit, also expressed in ohms, and represented by the symbol X.

In the same circuit, if φ is the angle of phase difference between the voltage and current, φ is defined by the following relations

$$\tan \varphi = \frac{2\pi f L - \frac{1}{2\pi f C}}{R} = \frac{X}{R}$$

$$\sin \varphi = \frac{2\pi f L - \frac{1}{2\pi f C}}{\sqrt{R^2 + \left(2\pi f L - \frac{1}{2\pi f C}\right)^2}} = \frac{X}{Z}$$

$$\cos \varphi = \frac{R}{\sqrt{R^2 + \left(2\pi f L - \frac{1}{2\pi f C}\right)^2}} = \frac{R}{Z}.$$

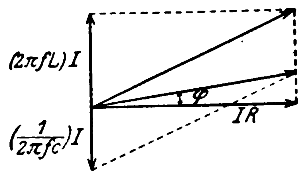


Fig. 244.

These relations are plainly shown in the vector diagram, fig 244. The actual power of the circuit we have considered so far is represented by

$$P = EI \cos \varphi$$

The effect of the inductive reactance $2\pi f L$ is to cause the currrent to lag behind the e. m. f., whereas the condensive reactance $\frac{1}{2\pi f C}$ causes a leading current. As the two effects oppose each other, there will be a condition where one effect will be neutralized by the other. This is called resonance and is expressed by

$$2\pi f L = \frac{1}{2\pi f C} \quad \text{or} \quad 2\pi f L - \frac{1}{2\pi f C} = 0.$$

As will be seen, $\varphi = 0$, in other words the current gets in phase whith the e. m. f. The condition of maximum current is attained, that is $I = \frac{E}{R}$.

In transmission lines, the inductive reactance is produced because of the cutting by the wire of the lines of force set up by the current in said wire. The condensive reactance is created on account of the wires acting in the same manner as condenser plates, the dielectric medium between them being the air.

The capacity of one wire to neutral is represented by

$$C = \frac{38.6}{10^3 \log_{10} \frac{2D}{d}}, \text{ m.f. per mile.}$$

The capacity between 2 wires will be $\frac{C}{2}$. It will be also important to remember that charging current of any condenser is given by

$$I_C = \frac{2\pi f C E}{10^6} \quad \text{or} \quad I_C = \frac{E}{\frac{10^6}{2\pi f C}} = \frac{E}{X_C}.$$

The self inductance per wire has the following expression:

$$L = \frac{0.161}{10^3}\left[2 \log_C \frac{2D}{d} + \frac{1}{2}\right], \text{ in henries per mile.}$$

In the above

C = Capacity between conductor and neutral plane,
D = Distance between outside of conductor and neutral plane, inches,
d = diameter of wire, inches;
L = self-inductance per wire, in henries per mile:

In the symbolic method, the respective values[1] of the factors influencing alternating current circuits are:

Resistance $= R$

Inductive reactance $= 2\pi f L = X_L = j x_L$

Condensive reactance $= \frac{1}{2\pi f C} = X_C = -j x_C$

Reactance $= 2\pi f L - \frac{1}{2\pi f C} = j(x_L - x_C) = \pm j x$

Impedance $= Z = \sqrt{R^2 + x^2} = R \pm j x$.

For convenience in calculations, the following notation is frequently used:

$$p = \cos\varphi = \frac{R}{Z} \quad \text{and} \quad q = \sin\varphi = \frac{X}{Z}.$$

Other functions which facilitate the calculations of alternating current circuits are as follows:

$$g = \text{conductance} = \frac{R}{Z^2} = \frac{R}{(R \pm j x)^2}$$

$$b = \text{susceptance} = \frac{X}{Z^2} = \frac{X}{(R \pm j x)^2}$$

$$Y = \text{admittance} = \frac{1}{Z} = \frac{1}{R \pm j x}.$$

The capacity of a transmission line is considered in various ways. For small lines, it is sufficient to assume that the total capacity is shunted across the line at the middle; Perrine and Baum have shown that the error attained in such cases is very small. With high voltages and long distances, the problem of distributed capacity is to be considered.

It has also been customary to divide the imaginary condenser in two parts, and suppose each part to be shunted across the conductors at each end. Still, another way is to assume the condenser as being divided in

[1] When refering to current, $+j$ means leading current or anti-inductive load. When referring to tension, $+j$ means leading volts or inductive load.

six parts, one sixth being supposed connected across at each end, and two thirds at the middle. This method has been recommended by Dr. Steinmetz. There is no doubt that an error is introduced by the use of these methods, but the actual condition of uniformly distributed capacity cannot be analyzed except by means of the hyperbolic functions.

It is well to bear in mind, when treating with alternating current circuits, that this sort of transmission line must be considered as a circuit having resistance and inductive reactance in series with the load and a condensive reactance in parallel with the load. The capacity of stranded conductors being greater than that of solid conductor, the outside diameter of the cable is taken as the basis for the calculations.

138. Surges in Transmission Lines.

Electrical surges may be due to several causes: static charges, high frequency atmospheric discharges and the sudden opening or closing of a line.

Static charges may originate when a loaded cloud comes close to the line, and a considerable rise in voltage can result.

High frequency atmospheric discharges are generally produced when lightning occurs between two clouds. A wave will be formed in the conductor (fig. 245) which will reach the transformer T.

The inductance of the latter will offer a considerable resistance to the high tension current, and the excess of this voltage will be distributed over the first windings of the transformers, obviously producing damage in them. A protection against static charges may be made by inserting high resistances between the line and the ground. Such apparatus is described in the paragraph relative to lightning arresters.

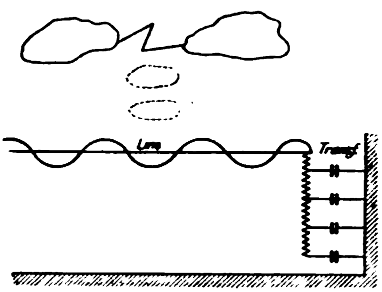


Fig. 245.

Surges due to sudden opening or closing of switches will now be examined. Let

L = inductance of the line,
I = current of the line,
C = capacity of the line,
V = voltage rise.

If the line is suddenly interrupted, the magnetic energy stored in the line will be $\frac{L I^2}{2}$; if C is the capacity of the line, the corresponding tension V will be found by the relation

$$\frac{L I^2}{2} = \frac{C V^2}{2} \text{ or } V = I \sqrt{\frac{L}{C}}.$$

The term $\sqrt{\frac{L}{C}}$ is constant for any line, and its value is usually taken as 200, so that the value of the voltage is determined by

$$V = 200\, I.$$

It is seen therefore, that the calculations for this case are very simple, a thing that is not true for the calculation of surges due to the closing of the line at its far end. The location of a transformer at this point complicates the analysis of the phenomena. The saturation of the iron core must be considered and the capacity of the transformer is to be appreciated.

139. High-Voltage Disturbances in Transmission Lines.

a) Resonance.

A transmission line is fundamentelly a circuit as represented by fig 246. It has been stated on page 257 that the capacity effect is opposed to the inductance effect. If both effects neutralize each other, resonance occurs and nothing but the resistance of the line would be opposed to the current. Now let us examine what may happen practically.

Suppose a line of relatively small resistance, but with a high capacity, if a high inductance is placed in series, a dangerous voltage can be produced.

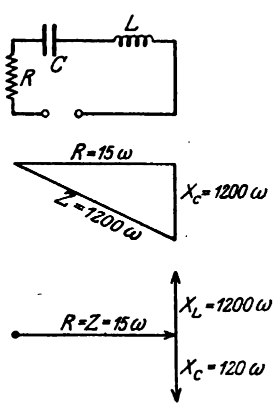


Fig. 246.

For instance, let

$$E = 30000 \text{ volts}$$
$$R = 15 \text{ ohms}$$
$$X_C = 1200 \text{ ohms}.$$

Then

$$Z = \sqrt{15^2 + 1200^2} = 1200 \text{ in round figures.}$$

Therefore,

$$I = \frac{E}{Z} = \frac{30000}{1200} = 25 \text{ amperes.}$$

If there is resonance in the circuit, and we have originally

$$E = 30000 \text{ Volts}$$
$$X_L = 1200 \quad \omega$$
$$X_C = 1200 \quad \omega$$
$$R = Z = 15 \ \omega,$$

then

$$I = \frac{E}{Z} = \frac{30000}{15} = 2000 \text{ amperes.}$$

The tension at the capacity and inductive reactances would be high as the following conditions show:

$$E_C = I X_C = 2000 \times 1200 = 2400000 \text{ volts}$$
$$E_L = I X_L = 2000 \times 1200 = 2400000 \text{ volts.}$$

A combination of partial resonance, with over-excitation and racing of prime mover, may also cause disturbances.

This can happen as in the case of a sudden short circuit or the use of a great amount of power at the load-end of the transmission line. In effect, in such a case, the automatic regulator maintains the original voltage by increasing the field excitation of the generator to a maximum.

The speed of the turbo-generator set is maintained because the turbine governor opens the gates. All this will last until the short-circuit or great overload is cut-off. Then suddenly, the excitation is for above normal, so is the speed of the turbo-generator set, and a rise of tension occurs in the line; this is plain because of the fact that in high tension transmission lines, the $I X_C$ factor represents about $50\,^0/_0$ of full load current.

The excess of voltage in the case just described is liable to reach as high as $100\,^0/_0$ or more above normal. If synchronous machines are used at the load end, conditions will be favorable in preventing this class of disturbance.

Finally, higher harmonics of the generator wave can produce resonance. It is well known that inductive reactance increases with an increase of frequency, whereas condensive reactance decreases with an increase of frequency. Therefore, at higher frequencies, they approach each other in magnitude. Higher harmonics may originate in transformers and in synchronous machines. In transformers, both the e. m. f. and exciting current cannot be sinewaves, because the exciting current of transformers depends on the magnetism by hysteresis cycle.

The constant current harmonic of transformers can be eliminated by a short circuit as is produced by a Δ-connection on one side of the transformer.

In the case of generators, and synchronous machines, the higher harmonics differ from those just described above in that they are of the constant potential type. When a generator is under load, the terminal voltage is the result of induced e. m. f. and the e. m. f. consumed by the reactance of the armature circuit. As this varies with double frequency, the result is a pulsation producing a third harmonic, which is the more common and prominent in synchronous machines.

140. Corona Effect.

If the potential across the air space separating two parallel transmission line wires exceeds what is called the critical voltage, the air becomes luminous or it breaks down. This phenomena which has received the name of corona, has a considerable heating effect, resulting in an increase of the conductivity of the air and involving a serious power loss in high tension transmission lines.

It has been observed that corona loss varies with the voltage, size of conductor, spacing of conductors, atmospheric pressure and condition of atmosphere. A decrease of pressure in the surrounding dielectric causes an increase in corona loss, and when the pressure is as low as 20 inches, the loss is considerable.

Transmission lines of high voltages make the consideration of corona losses an important factor, and the following formulae, developed by Mr. Peek[1], will be found useful for calculating the kilowatt loss for any transmission line.

[1] F. W. Peek jr. The Limiting Effect of Corona on the Electrical Transmission of Energy at High Voltages. General Electric Review. October 1911.

Let:

e_v = the "effective visual critical" voltage to neutral (in K. V.),
e_0 = "disruptive critical voltage" to neutral (in K. V.),
p = the total power loss in kilowatts per mile (single conductor),
e = effective kilovolt to neutral (applied),
k = a constant = 552,
g_0 = stress of the dielectric = 53.6 K. V. per inch effective,
δ = air density factor $= \frac{17.91\,b}{459 + t}$,
= s at 77° F. and 29.92 barometric pressure,
b = barometric pressure in inches,
t = degrees temperature Fahr.,
r = radius of conductor in inches,
s = distance between conductors in inches,
f = frequency,
m_0 = irregularity factor = 0.98 to 0.93 for weathered wires,
m_v = 0.72 local corona all along cable,
= 0.82 decided corona all along cable.

The "effective visual voltage" is given by the relation

$$e_v = 2.302\, m_v g_0 \delta r \left[1 + \frac{0.189}{\sqrt{r}}\right] \log_{10} \frac{s}{r}.$$

The "disruptive critical voltage" is given by

$$e_0 = 2.302\, m_0 g_0 \delta r \log_{10} \frac{s}{r}.$$

The total power loss in fair weather in kilowatts per mile of single conductor is determined by

$$p = \frac{k}{\delta} f \sqrt{\frac{r}{s}} \left[e - 2.302\, m_0 g_0 \delta r \log_{10} \frac{s}{r}\right]^2 \times 10^{-5} \text{ kilowatts.}$$

In case the effect of snow storms is considered, e_0 is to be lowered to 80% of its fair weather value, although it must be noted that such storms occur only at intervals.

The above formulae may also be useful in determining the size of hole should the wire be carried through a wall.

Prof. B. Davis, of Columbia University, has developed the following formula for determining the electrical intensity at the surface of a conductor that will just produce corona:

$$\frac{K}{r} = 3.8\,\delta\left(\frac{X_s}{\delta X_0} - 1\right) + 107\,\delta\left(\frac{X_s^2}{\delta^2 X_0^2} - 1 - 2\frac{X_s}{\delta X_0} \log \frac{X_s}{\delta X_0}\right)$$

where K = 4.3 for parallel transmission lines,
r = radius of conductor, in cm,
δ = density of air, usually taken as unity for atmospheric pressure,
X_s = gradient or electrical intensity at surface of conductor,
X_0 = 26,600 volts per centimeter.

141. Disruptive Distances.

The curves shown in the diagram, fig. 247, and determined by Mr. Vogelsang after numerous experiments, give

a) exploding distances in inches between horns or antennae in air,
b) exploding distances between points in air,
c) exploding distances between plates in air,
d) exploding distances between points in natural mineral oil such as is generally used for switches.

The current used in the experiments was alternating, 42 cycles, and nearly of sinusoidal wave form. When a plate of glass is located parallel to the horns, and covered with a slight layer of metal, the curve *b* is obtained, as between points. This may be explained by the condensation of the plate, which gives to the tension curve a pointed form. The curve *c* is relative to the distance between copper plates, about 2 inches in diameter.

The curve corresponding to the disruptive distance in oil was determined by pointed electrodes; however the experiments tended rather to show that the form of the electrodes when submerged in oil had such little influence over the results that this feature could be neglected entirely. The safe distance between apparatus may be taken as that given in the curve *a* between the antennae and that of curve *d* in oil.

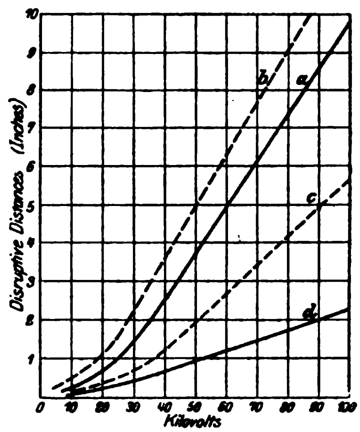


Fig. 247.

Table LX gives the disruptive distances in air between opposed sharp needle points for various effective sinusoidal voltages in inches and in centimeters. Comparing with the curves in fig. 264 it will be seen that the values given in the table correspond approximately to the values of curve *a*.

Table LX. *Sparking distances.*[1]

Kilovolts; square root of mean square	Distance		Kilovolts square root of mean square	Distance	
	Inches	Centimeters		Inches	Centimeters
5	0.225	0.57	140	13.950	35.4
10	0.470	1.19	150	15.000	38.1
15	0.725	1.84	160	16.050	40.7
20	1.000	2.54	170	17.100	43.4
25	1.300	3.30	180	18.150	46.1
30	1.625	4.1	190	19.200	48.8
35	2.000	5.1	200	20.250	51.4
40	2.450	6.2	210	21.300	54.1
45	2.950	7.5	220	22.350	56.8
50	3.550	9.0	230	23.400	59.4
60	4.650	11.8	240	24.450	62.1
70	5.850	14.9	250	25.500	64.7
80	7.100	18.0	260	26.500	67.3
90	8.350	21.2	270	27.500	69.8
100	9.600	24.4	280	28.500	72.4
110	10.750	27,3	290	29.500	74.9
120	11.850	30.1	300	30.500	77.4
130	12.900	32.8			

142. Loading and Transposition in Telephone Lines.

If two telephone wires are exposed to the inductive effect of a transmission line, they become center of induced currents, interfering with the action of the telephone line in a degree directly proportional to the difference in action of the inducting lines; if the latter are unequally distant from the telephone line. This trouble may be overcome by transpositions effected at regular intervals (fig 248), the result being that the inductive influences are neutralized. Call L_1 and L_2 the coefficients of self-induction per unit length between two telephone wires and the wire of a transmission line having a current I, the electro-motive forces induced in a length l, are respectively:

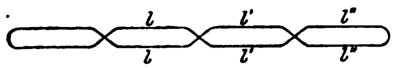


Fig. 248. Transposition in Telephone Lines.

$$E_1 = -lL_1 \frac{dI}{dt}$$

$$E_2 = -lL_2 \frac{dI}{dt}.$$

The resulting electromotive force acting in the telephone circuit will then be

$$E = E_1 - E_2 = l(L_2 - L_1)\frac{dI}{dt}.$$

If the telephone wires are equally distant (fig. 249), the coefficients of self-induction are equal and the value of the induced e. m. f. is zero.

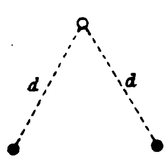


Fig. 249.

However, if such distances are not equal, as is always the case in three phase transmission lines it is necessary to effect transpositions of the lines.

In the latter case, the electro-motive forces will have for values:

$$E_1 = -(lL_1 + l'L_2)\frac{dI}{dt}$$

$$E_2 = -(lL_2 + l'L_1)\frac{dI}{dt}$$

and the resulting e. m. f. would be:

$$E_1 - E_2 = (L_1 - L_2)(l' - l)\frac{dI}{dt},$$

which becomes equal to zero for $l = l'$.

Mr. E. J. Berg[1], analyzes as follows the inductive effect between a transmission line and a telephone line (fig. 250), there being no transposition and no ground.

Let: A, B, C, represent the conductors of a three phase transmission line, and E and D, a telephone line, the diameters af all conductors being assumed equal. Call:

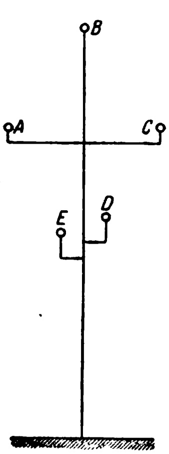


Fig. 250.

[1] E. J. Berg. Electrical Energy.

$a_1 =$ the distance AD
$a_2 =$ the distance AE
$b_1 =$ the distance BD
$b_2 =$ the distance BE
$c_1 =$ the distance CD
$c_2 =$ the distance CE
$L_a =$ coeff. of self induction per mile between D and E due to A,
$L_b =$ coeff. of self induction per mile between D and E due to B,
$L_c =$ coeff. of self induction per mile between D and E due to C.

The values of the respective coefficients in milhenrys per mile will be

$$L_a = \frac{161}{10^3}\left[2\log_e\frac{2a_2}{d} - 2\log_e\frac{2a_1}{d}\right] = \frac{322}{10^3}\log_e\frac{a_2}{a_1}$$

$$L_b = \frac{322}{10^3}\log_e\frac{b_2}{b_1}$$

$$L_c = \frac{322}{10^3}\log_e\frac{c_2}{c_1}.$$

There being a phase displacement of 120°, the resultant is

$$L_r = \sqrt{\left[L_b - \frac{1}{2}\left(L_a + L_c\right)\right]^2 + 0.75\,(L_a - L_c)^2}.$$

The reactance being $\frac{2\pi L_r}{10^3}$, and the induced e. m. f. $= \frac{2\pi f}{10^3} L_r \cdot I$, where f is the frequency and I the current in each main conductor. The induced e. m. f. is proportional to the density of the current, and this condition may be overcome by proper transposition.

M. Pleijel cites the case of a railway using alternating current at 15 000 volts and 25 cycles. The railway follows telegraph and telephone lines for $3\frac{1}{2}$ miles. On the telephone lines, only a slight sound was discernable when the trains started or stopped. The tension between these lines and earth was about 20 volts at one end. But on a telephone line carried on the same posts with the transmission line, the tension was found to be 5000 volts. The trouble was overcome by substituting for the lightning guards an induction coil placed between the two line wires, the middle being connected to earth. In this particular case, measurements had shown that 75% of the interfering currents were electro-static and 25% electromagnetic.

The value of transposition as regards electrostatic effects is not accounted for as in the case of self-induction; Mr. Berg shows that in using the formula

$$E_x = E\,\frac{\log_{10}\frac{l}{x}}{\log_{10}\frac{R}{x}}$$

where

$l =$ distance from end of conductor to ground,
$r =$ radius of conductor,
$E =$ potential difference to ground, and
$E_x =$ potential difference to ground at distance x from conductor.

The resultant voltage above ground for D (fig. 250) is:

$$E_D = \sqrt{[E_b - 0.5\,(E_a + E_c)]^2 + 0.75\,[E_a - E_c]^2}$$

in which the voltage due to A is:

$$E_a = E\,\frac{\log \frac{L_a}{a_1}}{\log \frac{L_a}{r}};$$

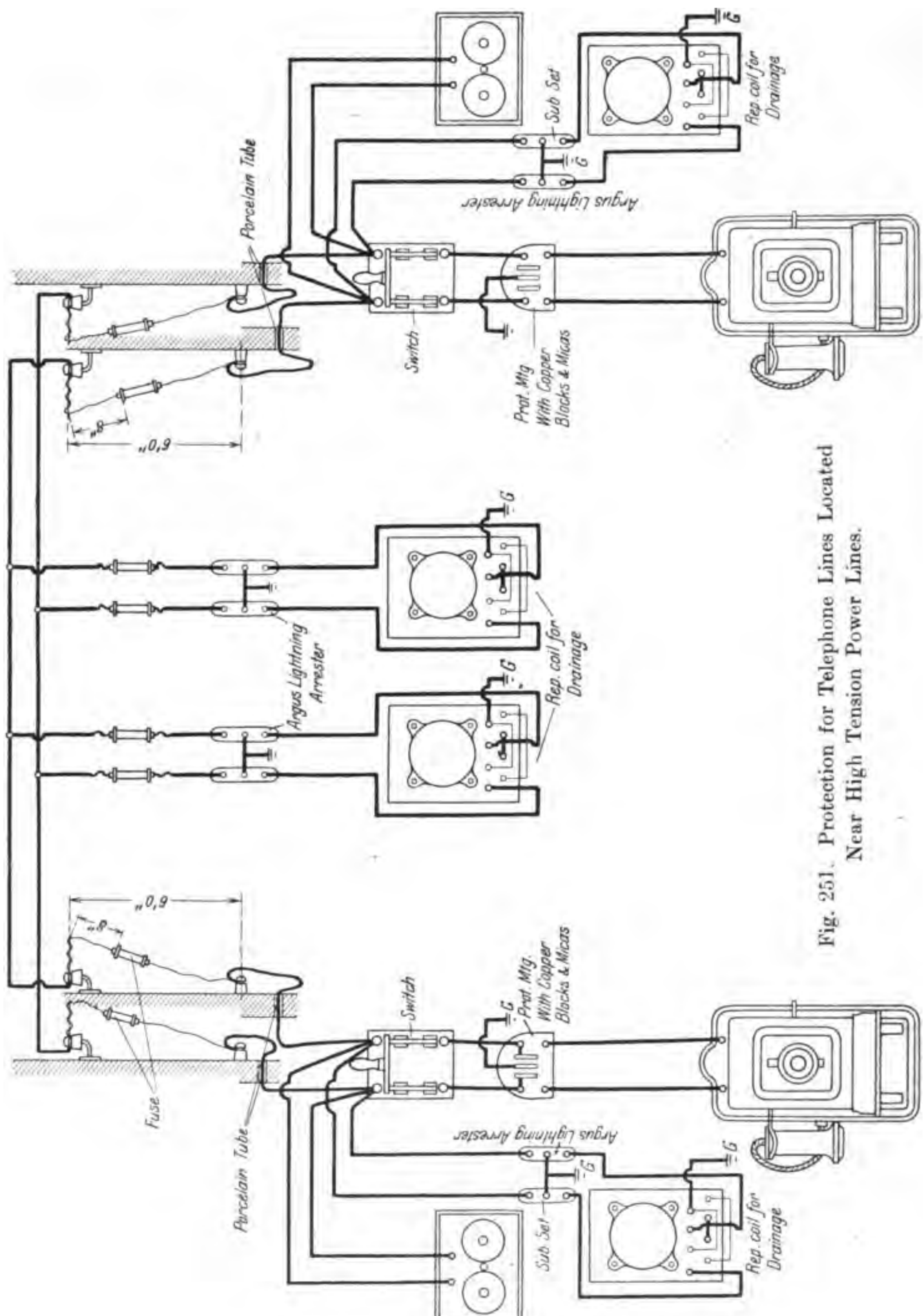


Fig. 251. Protection for Telephone Lines Located Near High Tension Power Lines.

E_b and E_c being determined similarly. In case of transmission lines using very high voltages, the static voltage may be considerable if the distance to the telephone line is small, the best remedy being to use a telephone transformer the secondary neutral of which has been grounded.

Fig. 251 shows Western Electric Co's practice for the protection of telephone lines located near high tension power lines.

143. Kelvin's Law.

Lord Kelvin has shown that the most economical conductor is that in which the annual cost of energy wasted is equal to the interest and depreciation on the capital outlay, i. e. proportional to the weight of the conductor. The cost of a transmission line may be represented by two parts:

1) the cost of the conductor and its supports and insulation;
2) the cost of labor involved in its installation.

The first item as well as the second can be considered as being composed of two parts; in the first case it is found that:

$$C = c + C_x A$$

in which

$c =$ constant cost per unit of length,
$C_x =$ variable cost per unit of length depending on A,
$A =$ area of cross-section of conductor, and
$C =$ cost per unit length of transmission line.

The second item corresponding to labor will give:

$$C' = c' + C_x' A .$$

In the following let

$r =$ rate of interest charged on total installation cost,
$d_c =$ rate of depreciation on cost of conductor material,
$d_e =$ rate of depreciation on cost of conductor supports,
$d_s =$ rate of depreciation on cost of station,
$C_{d'} =$ cost per annum of energy wasted in the line,
$C_{d''} =$ yearly depreciation and interest on cost of line,
$C_d = C_{d'} + C_{d''}$,
$N =$ number of hours of operation per year,
$C_e =$ cost of energy per kw. hour, dollars,
$C_s =$ cost of equipment of central station, per kw. dollars,
$I =$ current in amperes,
$A =$ cross-sectional area of conductor,
$\varrho =$ specific resistance of conductor,
$l =$ length of conductor,
$p,q =$ coefficients for convenience, and
$C_l =$ cost of the line.

The cost of the line will be

$$C_l = l\,[(c + C_x A) + (c' + C_x' A)]$$

and

$$C_{d''} = l\,[(c + C_x A)(r + d_c) + (c' + C_x' A)(r + d_e)]$$

which may be written

$$C_{d''} = p + qA$$

assuming that

$$p = l\,[c\,(r + d_c) + c'\,(r + d_s)]$$
$$q = l\,[C_x\,(r + d_c) + C_x'\,(r + d_e)]\,.$$

The resistance of the line is

$$R = \frac{\varrho\, l}{A}\,.$$

The energy lost in the line in Kw. hours will be denoted by:

$$W_l = \frac{\varrho\, l\, I^2 N}{1000 A}$$

which represents a sum of

$$C_1 = C_e W_l\,.$$

The annual capital charge in dollars for the power lost in the line is

$$C_2 = C_s \frac{\varrho\, l\, I^2}{1000\, A}\,(r + d_s)\,.$$

The cost in dollars of the lost energy will be:

$$C_{d'} = C_1 + C_2 = \frac{\varrho\, l\, I^2}{1000\, A}\,[N C_e + C_s\,(r + d_s)] = \frac{m}{A}\,.$$

Therefore:

$$C_d = C_{d'} + C_{d''} = \frac{m}{A} + p + qA\,.$$

This expression is a minimum when:

$$A = \sqrt{\frac{m}{q}}\,.$$

It will be noted that E, the voltage of the line, does not enter in the last equation, the section being therefore independent of the e. m. f., whereas m being a function of l, the section is influenced by the distance.

144. Calculation of Direct Current Transmission Lines.

In the following discussion let:

I = current in amperes,
E = voltage at end of line,
l = length of transmission line (one way),
e = voltage drop,
A = area of conductor in circ. mils,
N = number of horse power to be transmitted,
ϱ = resistance of copper wire per circ. mil foot,
W = weight of one circular-mil foot of copper,
C = cost of copper per circular mil foot,
K = cost of installation per horse power of generating plant,
C_t = cost of posts, labor, insulators etc., of transmission line,
C_r = cost of receiving station, per horse power,
β = efficiency of generators, and
α = efficiency of turbines.

The current will be given by:

$$I = \frac{746\,N}{E\alpha}$$

and the resistance of the line will have for value:

$$R = \frac{2\,\varrho l}{A}.$$

The voltage drop along the line is:

$$e = IR = \frac{746\,N}{E\alpha} \cdot \frac{2\,\varrho\,l}{A} = \frac{1492\,N\varrho\,l}{E \cdot \alpha \cdot A}.$$

The cross-section area of the conductor is found by the following equation:

$$A = \frac{1492\,N\varrho\,l}{E \cdot \alpha \cdot e}.$$

The cost of the line, two lengths, is equal to:

$$\frac{1492\,N\varrho\,l^2\,WC}{E \cdot \alpha \cdot e}$$

and the amount of generated power:

$$\frac{E+e}{E} \cdot \frac{N}{\alpha} \cdot \frac{1}{\beta}.$$

The cost of installation per horse power being K, the cost for N horse power will be NK and the expenses will be represented by:

$$\frac{E+e}{E \cdot \alpha \cdot \beta} NK.$$

The total expense per horse power will be:

$$\frac{E+e}{E \cdot \alpha \cdot \beta} K = \frac{1492\,\varrho l^2\,WC}{E\alpha C} + \frac{C_t}{N} + C_r.$$

The first cost, or cost of installation, will be a minimum for:

$$\frac{K}{\beta} = \frac{1492\,\varrho l^2\,WC}{e^2}$$

from which the voltage drop may be determined:

$$e = 38.6\,l \sqrt{\frac{\varrho\,WC\beta}{K}}$$

Calling m the efficiency of the line:

$$m = \frac{E \cdot \alpha \cdot \beta}{E+e}$$

or

$$e = \left[\frac{\alpha \cdot \beta - m}{m}\right] E$$

and the expenses in the terms of m are:

$$\frac{2K}{m} - \frac{K}{\alpha \cdot \beta} + \frac{Ct}{N} + C_r.$$

Some important facts are revealed by this analysis: the value of e for which the expenses are minimum is a function of l, and not of the e. m. f. of the machine, and the ratio between the cost of line and first cost of plant, depends upon the commercial efficiencies. The resistance of copper wire per circ.-mil foot at 75° Fahr. is equal to 5.25. For all practical purposes 2ϱ may be taken as 11 for copper and 16.5 for aluminum. Although these values are a little high, the error is on the safe side. If aluminum is used, C must be taken care of in that respect.

145. Calculation of Single Phase Transmission Line.

Let E_g = generator voltage,
E_r = voltage at receiving end of line,
$\omega = 2\pi f$, in which f = frequency,
R_2 = line resistance,
L_2 = self inductive coefficient of line,
I = current in amperes,
$\cos\varphi$ = power factor of the system at generator end, and
$\cos\varphi'$ = power factor of load at receiver end of line.

The cosine of the angle of lag is given by the equation:

$$\cos\varphi = \frac{R}{\sqrt{L^2\omega^2 + R^2}}$$

in which

R = resistance of the whole system; and
L = its coefficient of self-induction. The power factor of load at the receiving end of the line is generally from 0.60 to 0.80 for motor circuits, and 0.95 for lighting. In order to determine E_r, consider a system of rectangular co-ordinates as shown per fig. 252. Lay off an angle:

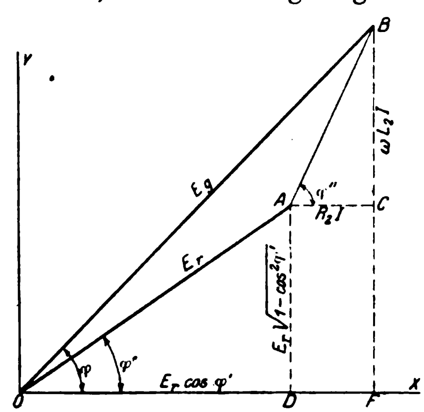


Fig. 252.

$AOX = \varphi'$ = angle of lag at receiving end of line and on OA measure OA = voltage at receiving end of line.

From A lay off IR_2 and $\omega L_2 I$ so that

$$\tan\varphi'' = \frac{\omega L_2}{R_2}.$$

The voltage at the generator will be given by OB and determined as follows:

$$E_g\cos\varphi = E_r\cos\varphi' + IR_2$$
$$E_g\sin\varphi = E_r\sqrt{1-\cos^2\varphi'} + \omega L_2 I \quad . \; . \; . \; . \; . \; (1)$$

Remembering that

$$\sqrt{1-\cos^2\varphi'} = \sin\varphi'$$

and substituting in (1) gives:

$$E_g\sin\varphi = E_r\sin\varphi' + \omega L_2 I.$$

From the triangle OFB, the value of E_g is found to be:

$$E_g^2 = [E_r \cos\varphi' + I R_2]^2 + [E_r \sqrt{1-\cos^2\varphi'} + \omega L_2 I]^2$$

or

$$E_g = \sqrt{[E_r \cos\varphi' + I R_2]^2 + [E_r \sin\varphi' + \omega L_2 I]^2}\,.$$

The efficiency of the line will be given by:

$$\varepsilon = \frac{E_r \cos\varphi'}{E_r \cos\varphi' + I R_2}\,.$$

If the efficiency ε is too low for good regulation, it is better to install a duplicate circuit, carrying each one half of the load. It is also the practice to use such apparatus at the receiving end of the line, as will increase the power factor. It will be noted in the above discussion, that the effect of capacity has been neglected. This effect, however, must be considered in the case of long transmission lines.

146. Single Phase Transmission Line. Economics.[1]

Let: E_r = voltage at receiving end of line,
P_g = kilowatt output at generator end,
l = length of line in miles (one way),
C_g = cost per-kw. year at generator station (dollars),
C = cost of conductor per pound (dollars),
p = interest rate on cost of conductor,
ϱ = resistance of copper wire per circ.-mil per mile, in ohms,
W = pounds per mil-mile of conductor,
R = total resistance of line (two ways) in ohms,
A = area of conductor, circular mils,
n = loss in terms of generated power,
$\cos\varphi$ = power factor of load,
m = annual cost per generated kilowatt and
K = a symbol for convenience.

The loss along the line has for expression:

$$P_g n,$$

The annual cost of loss along the line is determined by:

$$C_g P_g n$$

The weight of the conductors (both ways) is

$$2\,l\,W A$$

The cost of conductor will be represented by:

$$2\,C\,l\,W A$$

The interest on conductors per year will be represented by:

$$2\,p\,C\,l\,W A$$

The total resistance of the line is

$$R = \frac{2\,\varrho\, l}{A}$$

[1] For a full discussion of this subject, see Transmission Line Calculations, M. W. Franklin; The General Electric Review, March-April, 1910.

The drop along the line is determined by

$$\frac{n E_r \cos\varphi}{(1-n)} = \frac{1000 P_g R (1-n)}{E_r \cos\varphi}.$$

Substituting $R = \frac{2\varrho l}{A}$ in the above equation gives:

$$\frac{n E_r \cos\varphi}{(1-n)} = \frac{2000 P_g \varrho l (1-n)}{E_r A \cos\varphi}.$$

The conductor area will then be found to be:

$$A = \frac{2000 P_g \varrho l (1-n)^2}{E_r^2 n \cos^2\varphi} \quad \ldots \ldots \ldots \ldots (1)$$

The annual cost of line and interest charges are:

$$C_g P_g n + 2 p C W l A.$$

The annual cost per generated kilowatt is

$$m = \frac{C_g P_g n + 2 p C W l A}{P_g}.$$

Substituting for A its value given in (1) gives:

$$m = \frac{C_g P_g n + 2 p C W l \left[\frac{2000 P_g \varrho l (1-n)^2}{E_r^2 n \cos^2\varphi}\right]}{P_g}.$$

Expanding and simplifying

$$m = C_g n + \frac{K (1-n)^2}{E_r^2 n}$$

in which

$$K = \frac{4000 p C \varrho W l^2}{\cos^2\varphi}.$$

The annual cost per generated kilowatt will be a minimum for

$$n = \sqrt{\frac{K}{C_g E_r^2 + K}} \quad \ldots \ldots \ldots \ldots (2)$$

If the receiving end of the line is considered and calling

P_2 = kilowatts delivered, and
n = loss in terms of delivered power,

expressions (1) and (2) will become respectively:

$$A = \frac{2000 P_r \varrho l}{E_r^2 n} \quad \ldots \ldots \ldots \ldots (3)$$

$$n = \frac{1}{E_r} \sqrt{\frac{K}{C_g}} \quad \ldots \ldots \ldots \ldots (4)$$

147. Three Phase Transmission Line (Short Distance).

Let E_{eff} = virtual value of e. m. f.,
I_{eff} = mean virtual current per line,
P_g = power to be transmitted, or power at generator end, in watts,
$\cos\varphi$ = power factor,

$l =$ length of transmission line (one way) in feet,
$e =$ voltage drop in %,
$A =$ area of conductor in circular-mils, and
$R =$ resistance of the line (one way).

The power at the generator terminals is

$$P_g = E_{eff} I_{eff} \sqrt{3} \cos\varphi .$$

Considering the resistance r

$$R I_{eff} \sqrt{3} = \frac{E_{eff} \cos\varphi \cdot e}{100} .$$

The value of R at 75° Fahr. is given by

$$R = \frac{8.25\, l}{A} \text{ for aluminium}$$

$$R = \frac{5.50\, l}{A} \text{ for copper.}$$

Considering the case of copper and substituting:

$$\frac{5.50\, l}{A} I_{eff} \sqrt{3} = \frac{E_{eff} \cos\varphi \cdot e}{100} .$$

Multiplying both terms by $E_{eff} \cos\varphi$:

$$\frac{5.50\, l}{A} I_{eff} E_{eff} \cos\varphi \sqrt{3} = \frac{E^2_{eff} \cos^2\varphi \cdot e}{100}$$

or

$$\frac{E^2_{eff} \cos\varphi \cdot e}{100} = \frac{5.50\, l}{A} \sqrt{3} P_g .$$

The area of conductor is then found to be:

$$A = \frac{5.50\, l \sqrt{3}\, P_g}{E^2_{eff} \cos^2\varphi \cdot e} = \frac{940\, l\, P_g}{E^2_{eff} \cos^2\varphi \cdot e} .$$

148. Three Phase Transmission Line. Economics.[1]

Let $P_g =$ Output at generator end, in watts;
$P_r =$ watts delivered,
$E_g =$ voltage at generator end,
$E_r =$ voltage at receiving end,
$I =$ effective current in each wire,
$R =$ total resistance of line (one way),
$l =$ length of line,
$\varrho =$ specific resistance of conductor,
$k =$ relative loss,
$W =$ specific weight of conductor,
$A =$ conductor area,
$C =$ cost of conductor per unit weight,
$C_t =$ cost of posts, insulators, labor, etc., for transmission line, per unit length,

[1] See foot-note of paragraph 146.

$d =$ rate of depreciation,
$k =$ cost of a watt-year, and
$\cos\varphi =$ power factor of load.

The loss along the line has for value:

$$\varrho = 3 I^2 R = \frac{\varrho l P_r^2}{3 A E_r^2 \cos^2\varphi}.$$

The annual cost of loss along the line is

$$\frac{K \varrho l P_r^2}{3 A E_r^2 \cos^2\varphi}.$$

The cost of the transmission line is represented by:

$$l(3\,WCA + C_t).$$

Depreciation is given by the expression

$$dl(3\,WCA + C_t).$$

The problem consists in making a minimum the expression:

$$dl(3\,WCA + C_t) + \frac{K \varrho l P_r^2}{3 A E_r^2 \cos^2\varphi}.$$

Differentiating with respect to A, and equating to zero, gives

$$9\,d\,WCE_r^2 \cos^2\varphi - K \varrho P_r^2 A^{-2} = 0.$$

Solving with respect to A gives:

$$A = \frac{P_r}{3\,E_r \cos\varphi} \sqrt{\frac{\varrho K}{d\,WC}}.$$

The relative loss is given by

$$k = \frac{3\,I^2 R}{3\,E_r I \cos\varphi + 3\,I^2 R} = \frac{I R}{E_r \cos\varphi + I R}.$$

We have

$$P_r = 3\,E_r I \cos\varphi \quad \text{and} \quad R = \frac{\varrho l}{A}$$

also

$$A = \frac{(1-k)}{k} \cdot \frac{\varrho l P_r}{3\,E_r^2 \cos^2\varphi} \quad \ldots\ldots\ldots (1)$$

But P_g being known, we may write

$$P_r = P_g(1-k).$$

Substituting for A, its value in (1), gives:

$$k = \frac{\varrho l}{l + E_r \cos\varphi \sqrt{\dfrac{K}{d\,WC}}}$$

from which

$$A = \frac{k P_g}{3\,d\,WC\left[l + E_r \cos\varphi \sqrt{\dfrac{K}{d\,WC\varrho}}\right]}.$$

Considering that in a three phase transmission system, the area of each of the three wires is equal to one half the area used in an equivalent single phase system, the conductor cross section and economic loss may be derived directly from the expressions (3) and (4) (paragraph 145) substituting $\frac{3}{4}K$ for K in said expressions.

Thus

$$A = \frac{1000\,P_g \varrho l\,(1-n)^2}{E_r^2\, n \cos^2\varphi} \quad \ldots\ldots\ldots (2)$$

$$n = \sqrt{\frac{3K}{4C_g E_r^2 + 3K}} \quad \ldots\ldots\ldots\ldots (3)$$

in which the symbols are as in paragraph 153.

If the receiving end is considered, equations (2) and (3) become

$$A = \frac{1000\, P_r \varrho l}{E_r^2 n \cos^2 \varphi} \quad \ldots\ldots\ldots\ldots (4)$$

$$n = \frac{0.866}{E_r} \sqrt{\frac{K}{C_g}} \quad \ldots\ldots\ldots\ldots (5)$$

149. Single Phase Transmission Line Calculations, Symbolic Method.

In former paragraphs on transmission line calculations, condensive and inductive reactance effects have been neglected. It will be of interest to point these out and a practical problem is now presented.

Let it be desired to transmit electric energy over an transmission line 100 miles in length. The current is to be single phase, and of a frequency of 60 cycles per second. The tension at the substation is to be 110000 volts. The line wires are No. 000 B & S gauge, to be placed 96 inches apart. The substation input is to be 30000 kilowatts: It is required to

a) find the capacity and charging current, with no load.

b) determine the resistance, inductance and inductive reactance.

c) find the voltage at the power house for 110000 volts at the substation, where an inductive load of 30000 kilowatts with cos $\varphi = 0.90$ will be utilized.

d) find the same as indicated in (c) with cos $\varphi = 0.90$, anti-inductive.

The problem is solved as follows:

a) The capacity between a conductor and the neutral plane is given on page 257.

The diameter of No. 000 B & S wire is 0.41 inch.

By substitution in said equation and remembering that $2D = [(96 \times 2) - 0.20]$ we obtain

$$c = \frac{38.6 \times 100}{10^3 \log_{10} \frac{191.8}{0.41}} = 1.45 \text{ microfarad.}^{1}$$

The capacity between conductors will then be:

$$\frac{1.45}{2} = 0.73 \text{ m.f.} = 0.000\,000\,73 \text{ farad.}$$

The condensive reactance is determined by

$$x_c = \frac{1}{2\pi f C} = \frac{1}{2 \times 3.14 \times 60 \times 0.000\,000\,73} = 3630 \text{ ohms.}$$

The value of the charging current at no load is

$$I_c = \frac{2\pi f C E}{10^6} = \frac{2 \times 3.14 \times 60 \times 0.000\,000\,73 \times 110\,000}{10^6} = 30.3 \text{ amperes.}$$

[1] All calculations have been made by means of the slide rule.

It is to be noted that in this particular case and with $\cos\varphi = 1$, the charging current represents 11.1 per cent of the full load current. This percentage may be decreased to a certain extent by increasing the distance between the wires.

b) The resistance of No. 000 B & S wire being 0.32649 ohms per mile, the total resistance will be:

$$R = 0.32649 \times 200 = 65.3 \text{ ohms.}$$

Inductance is calculated by means of the equation given on page 258; with proper substitution its value is:

$$L = \frac{0.161}{10^3}\left[2\log_e \frac{191.8}{0.41} + \frac{1}{2}\right] = 0.00206 \text{ henry per mile.}$$

The inductive reactance for 100 miles of transmission line wire will be

$$x_L = 2\pi f L = 2 \times 3.14 \times 60 \times 0.00206 \times 100 = 77.6 \text{ ohms}$$

or 155.2 ohms for the whole circuit.

c) For a clear understanding, the circuit as it actually is, has been represented in fig. 253.

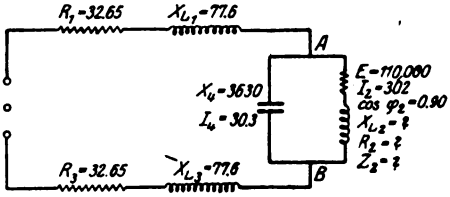


Fig. 253.

The fundamental equations to be considered are:

$$I_2 = \frac{E_2}{Z_2}; \quad p_2 = \frac{R_2}{Z_2}; \quad q_2 = \frac{X_2}{Z_2}.$$

By proper substitution, we obtain the following:

$$302 = \frac{110000}{Z_2}; \quad 0.90 = \frac{R_2}{Z_2}; \quad 0.425 = \frac{X_2}{Z_2} \text{ for } \varphi\, 25^\circ 10'.$$

Therefore:

$$Z_2 = 365; \quad R_2 = 328; \quad X_{L2} = 155.$$

Assuming the capacity to be in parallel with the load, the current circulating between A and B according to the Kirchhoff Law, will have the following value:

$$I_{24} = p_2 I_2 + p_4 I_4 + j q_4 I_4 - j q_2 I_2.$$

Substituting known values, we obtain

$$I_{24} = (0.90 \times 302) + (0 \times 30.3) + (j \times 30.3) - (j \times 0.425 \times 302)$$
$$= 272 - 98.1\, j$$
$$I_{24} = 289 \text{ amperes.}$$

Determination of power will be made as follows:

$$P_{24} = P_2 + P_4 = 30{,}000{,}000 + 0 = 30{,}000{,}000 \text{ watts,}$$
$$P_{24} = I_{24} \times E_{24} \times \cos\varphi_{24}$$

from which the power factor is obtained:

$$\cos\varphi_{24} = \frac{30{,}000{,}000}{289 \times 110000} = 0.946 \qquad (\varphi = 18^\circ 10')$$

The impedance is represented by

$$Z_{24} = \frac{110000}{289} = 381 \text{ ohms.}$$

Finally, the following relations are established:

$$R_{24} = p_{24} \times Z_{24} = 0.946 \times 381 = 360 \text{ ohms,}$$
$$X_{24} = q_{24} \times Z_{24} = 0.312 \times 381 = 119 \text{ ohms.}$$

The circuit, which we have here considered can now be represented in a simpler form, as is shown in fig. 254.

It is a series circuit, the solution of which offers no difficulty:

$$Z_0 = 65.30 + 360 - 119 j - 776.0 j$$
$$Z_0 = 505 \text{ ohms.}$$

It will be remembered that $E_0 = I_0 Z_0$, so by substitution we obtain:

$$E_0 = 289 \times 505 = 146000 \text{ volts.}$$

The regulation would now be

$$\frac{146000 - 110000}{110000} = 0.327 \text{ or } 32.7\,\%$$

which is higher than practice would allow.

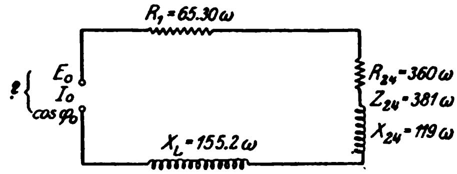


Fig. 254.

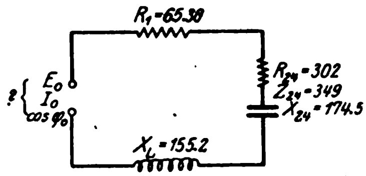


Fig. 255.

d) For anti-inductive load, we have the following:

$$I_{24} = p_2 I_2 + p_4 I_4 + j q_4 I_4 + j q_2 I_2$$
$$= 272 + 30.3 j + 128.4 j$$
$$I_{24} = 315 \text{ amperes.}$$

As before, $P_{24} = 30{,}000{,}000$ watts and

$$P_{24} = I_{24} \times E_{24} \times \cos\varphi_{24}$$

from which

$$\cos\varphi_{24} = \frac{30000000}{315 \times 110000} = 0.866 \qquad (\varphi_{24} = 30^0)$$

Therefore

$$Z_{24} = \frac{110000}{315} = 349 \text{ ohms.}$$

Finally,

$$R_{24} = p_{24} Z_{24} = 0.866 \times 349 = 302$$
$$X_{24} = q_{24} Z_{24} = 0.500 \times 349 = 174.5.$$

The circuit may be laid out as per fig. 255.

We come to the final values:

$$Z_0 = 65.3 + 302 + 174.5 j - 155.2 j$$
$$= 367.3 + 19.3 j$$
$$Z_0 = 414 \text{ ohms.}$$

Now, as $E_0 = I_0 Z_0$, we have that

$$E_0 = 315 \times 414 = 130000 \text{ volts.}$$

The regulation in this case would be

$$\frac{130000 - 110000}{110000} = 0.1725 \text{ or } 17.25\,\%.$$

It is believed that the foregoing problem is fairly illustrative of conditions which can be met with in practice. It may be noted, however. that for simplicity in calculations, the capacity of the line has been assumed in parallel with the load and at the load. For refined results, the capacity is considered as distributed along the line. Naturally, in a commercial proposition, the neutralizing effect of capacity and inductance would be considered at the same time, and the regulation would be greatly improved.

The symbolic method may also be applied in the following way for the solution of single phase transmission line problems.[1]

Suppose a transmission line of impedance $\dot{z} = r + jx$ delivers power to a receiver circuit of admittance $\dot{Y} = g - jb$ at a constant voltage E. If the capacity of the line is assumed concentrated at the middle, determine the charging current of the line, the total current delivered by the generator and the terminal volts of the generator.

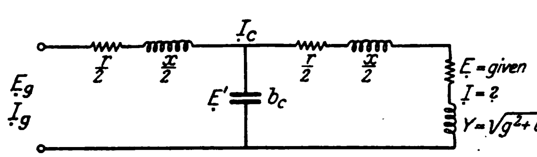


Fig. 256.

The circuit may be laid out diagramatically as per Fig. 256.

The condensive susceptance is

$$y_c = b_c = \frac{1}{x_c}$$

and is represented as a condenser across the line.

The current in the receiver circuit is

$$\dot{I} = \dot{E}(g - jb)$$

and the e. m. f. at the center of the line is

$$\dot{E}' = \dot{E} + \dot{I}\left(\frac{r + jx}{2}\right) = \dot{E}\left[1 + \frac{r + jx}{2}\left(g - jb\right)\right].$$

The charging current of the line is

$$\dot{I}_c = \dot{E}' j b_c = j b_c \dot{E}\left[1 + \frac{r + jx}{2}\left(g - jb\right)\right]$$

and the current at the generator is

$$\dot{I}_g = \dot{I} + \dot{I}_c = \dot{E}\left[g - jb + jb_c\left\{\frac{r + jx}{2}\left(g - jb\right)\right\}\right] \quad . \; . \; . \; . \; . \; (1)$$

The terminal e. m. f. of the generator is

[1] Clarence V. Christie. Electrical Engineering — Page 512.

$$E_g = E' + I_g\left(\frac{r + jx}{2}\right)$$
$$= E\left\{1 + \frac{r+jx}{2}(g - jb) + \frac{r+jx}{2}(g - jb) + jb_c\left(\frac{r+jx}{2}\right)\right.$$
$$\left. + jb_c\frac{(r+jx)^2(g-jb)}{4}\right.$$
$$= E\left\{1 + (r+jx)\left(g - jb + j\frac{b_c}{2}\right) + j\frac{b_c}{4}(r+jx)^2(g-jb)\right\} \quad (2)$$

In the case of lines of small capacity, the last term may be neglected and equation (2) becomes

$$E_g = E\left[1 + (r+jx)\left(g - jb + j\frac{b_c}{2}\right)\right] \quad . \; . \; . \; . \; . \; (3)$$

This is equivalent to replacing the capacity susceptance b_c at the center of the line by a condenser of susceptance $\frac{b_c}{2}$ at the receiver end of the line.

150. Three-Phase Transmission Line Calculations, Symbolic Method.

The electric energy to be received at the substation will consist of 30000 K.V.A. three-phase current at 60 cycles per second.

The tension at the substation is to be 110000 volts. The line wires are N° 4-0 B. & S. gauge, to be placed 96 inches apart. The power factor at the substation is to be $\cos\varphi = 0.90$:

a) find the capacity and charging current at no load ($\cos\varphi = 1$).

b) determine the resistance, inductance and inductive reactance.

c) find the resistance and reactance drops in each conductor.

d) find the capacity susceptance of each conductor to neutral, assumed concentrated at the middle of line; find also the load admittance, load conductance and load susceptance per phase.

e) determine the voltage, current and power factor at the generating station.

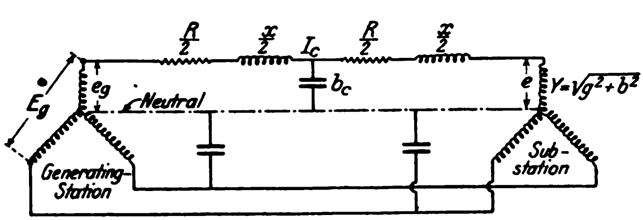


Fig. 257.

The circuit will be laid out as per fig. 257. It is to be remembered that

$$e = \frac{110000}{\sqrt{3}} = 63600 \text{ volt}$$

and the load current per conductor has the following value

$$I = \frac{30{,}000{,}000}{\sqrt{3} \times 110000} = 158 \text{ amperes.}$$

The problem is then solved as follows:

a) The equation for capacity between a conductor and the neutral plane is given on page 257.

The diameter of N° 4-0 B. & S. wire being 0.46 inch, by substitution in said equation, and considering that $2D = [(96 \times 2) - 0.23] = 191.77$ inches, we obtain

$$C = \frac{38.6 \times 100}{10^3 \log_{10} \frac{191.8}{0.46}} = 1.47 \text{ microfarad.}$$

The condensive reactance between a conductor and the neutral is

$$X_c = \frac{1}{2\pi f C} = \frac{10^6}{2 \times 3.14 \times 60 \times 1.47} = 1800 \text{ ohms.}$$

The value of the charging current at no load is

$$I_c = \frac{2\pi f C e}{10^6} = \frac{2 \times 3.14 \times 60 \times 63600 \times 1.47}{10^6} = 35.3 \text{ amperes.}$$

b) The resistance of N° 4-0 B. & S. (Mathiessen Standard) wire is 0.2667 ohms per mile. The total resistance will be

$$R = 0.2667 \times 100 = 26.67 \text{ ohms.}$$

The inductance will be found to be

$$L = \frac{0.161}{103}\left[2 \log_e \frac{191.8}{0.46} + \frac{1}{2}\right] = 0.00202 \text{ henry per mile.}$$

The inductive reactance for 100 miles will be

$$X_L = 2\pi f L = 2 \times 3.14 \times 60 \times 0.00202 \times 100 = 76.3 \text{ ohms.}$$

c) The resistance drop in each conductor will be

$$IR = 158 \times 26.67 = 4220 \text{ volts} = \frac{4220 \times 100}{\frac{110000}{\sqrt{3}}} = 6.64\,\%.$$

The reactance drop per conductor will be

$$I X_L = 158 \times 76.3 = 12050 \text{ volts} = \frac{12050 \times 100}{\frac{110000}{\sqrt{3}}} = 19\,\%.$$

Assume the resistance drop in the step-up and step-down transformer as being $\frac{1}{2}\,\%$ each, then the total resistance per line would be: $6.64 + \frac{1}{2} + \frac{1}{2} = 7.74\,\%$ and the equivalent resistance would represent

$$R' = \frac{26.67 \times 7.74}{6.64} = 31.1 \text{ ohms.}$$

Doing the similar operation for a $4\,\%$ reactance drop per transformer, we would obtain

$$\text{Reactance drop per line} = 19 + 4 + 4 = 27\,\%.$$

The equivalent reactance per line is

$$X_L = \frac{76.3 \times 27}{19} = 108 \text{ ohms.}$$

d) The capacity susceptance per line, assumed concentrated at the middle is

$$b_c = \frac{1}{X_c} = \frac{1}{1800} = 0.000555.$$

The load admittance per phase is

$$Y = \frac{I}{e} = \frac{158}{\frac{110000}{\sqrt{3}}} = 0.0025.$$

The load conductance per phase is as follows:

$$g = \frac{I}{e}\cos\varphi = Y\cos\varphi = 0.0025 \times 0.90 = 0.00225.$$

The load susceptance per phase is:

$$b = \frac{I}{e}\sin\varphi = Y\sin\varphi = 0.0025 \times 0.436 = 0.00109.$$

The circuit is then laid out as per fig. (258).

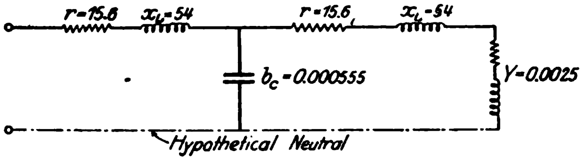


Fig. 258.

e) Now the voltage between the line and neutral at the generating station is indicated by equation 3 page 279, which, by proper substitution gives:

$$\dot{e}_g = \dot{e}\left[1 + (31.1 + 108j)\left(0.00225 - 0.00109j + \frac{0.000555}{2}\right)\right]$$

$$\dot{e}_g = \dot{e}[1.197 + 0.240j] = \frac{110000}{\sqrt{3}}\sqrt{\overline{1.197}^2 + \overline{0.24}^2} = 80500 \text{ volts};$$

therefore $E_g = 80500 \times \sqrt{3} = 139200$ volts.

The angle of lead over the terminal voltage is

$$\varphi' = \tan^{-1}\frac{0.240}{1.197} = 11^\circ 20'.$$

f) The current per conductor at the generating station is given by equation 1 page 278.

By proper substitution

$$\dot{I}_g = \dot{e}[0.00225 - 0.0010j]$$

$$= \frac{110000}{\sqrt{3}}\sqrt{\overline{0.00225}^2 + \overline{0.0010}^2} = 157 \text{ amperes.}$$

The current lags behind the terminal e.m.f. by an angle

$$\varphi'' = \tan^{-1}\frac{0.00100}{0.00225} = 23^\circ 55'$$

and finally, the current lags behind the generator voltage by an angle

$$\varphi_g = \varphi' + \varphi'' = 11^\circ 20' + 23^\circ 55' = 35^\circ 15'$$

$$\cos\varphi_g = 0.82.$$

The copper loss in the three wires has the following value:

$$3[I_g^2 + I^2]\frac{R}{2} = 3[\overline{157}^2 + \overline{158}^2] \times 15.6 = 2320 \text{ Kw.}$$

or in per cent

$$\frac{2320 \times 100}{30000 \times 0.82} = 9.45\,^0/_0.$$

This loss would be increased about $1\,^0/_0$ due to the iron losses in the transformers, so that the total loss would be $10.45\,^0/_0$. In other words, the efficiency of the transmission line would be $100 - 10.45 = 89.55\,^0/_0$.

The regulation is given as follows:

$$\frac{(139200 - 110000)}{110000} \times 100 = 26.6\,^0/_0$$

which is bad regulation, considered from a practical point of view.

If a commercial problem were here to be considered, a value of regulation not exceeding $10\,^0/_0$ would be substituted in the last expression, and the corresponding value of E_0 would first be determined. Upon this basis, the proper value of R, and appropiate size and spacing of wires would have to be determined.

151. Transmission Line Insulators.

The prime qualities of good insulators are:

1.—That they possess surfaces to which the rain cannot well get access, thus offering increased efficiency in stormy weather;

2.—That dust cannot collect in some protected place where it could not be dislodged by the wind;

3.—That they possess a mechanical resistance such that they can resist all the strains of the transmission line; and

4.—That no difficulty be encountered in their manufacture.

In his study of high voltage insulators, M. Friese established the following efficiency relations:

Calling:

$$A = \frac{\text{insulating value under rain}}{\text{insulating value under dry condition}}$$

$$B = \frac{\text{insulating value under rain}}{\text{weight of the insulator}}$$

Then $A \times B =$ insulating value.

Having tested comparatively the efficiency of a German insulator of the Hermsdorf Delta Glocke type and an American insulator that had practically the same value for A, M. Friese observed the following results:

Table LXI.

	Efficiency under rain Volts	Efficiency dry Volts	Weight Grams	A	B	Insul. value	Area of top shell Squ. cm.	Height cm.
German Delta Glocke	49,000	93,000	2125	0.53	23	12	240	19.5
American Insulator	49,000	93,000	2910	0.53	16.8	9	325	23.0

From table LXI, it is noted that an insulator of the German type, weighing 4.7 lbs., having a diameter of 7 inches, and being $7\frac{3}{4}$ inches high,

has nearly the same insulating value as an American insulator weighing 6.4 lbs., having a diameter of 8 inches and being 9 in. high.

These tests show that the Delta Glocke type of insulator is most efficient and as such is extensively used in Europe. This type has been manufactured up to 12 inches diameter, and it has the peculiarity of having but one thickness of porcelain between the line and the pin.

The underhanging insulator was designed as a result of the advantages of the Delta Glocke type. In designing underhung insulators, it is necessary to consider that they must be so constructed that in case the insulator is destroyed, the line will not fall to the ground.

M. K. Kuhlmann recommends using as low as possible an electric elementary capacity per unit of length, in order to eliminate the radiation and creeping sparks occurring in high tension line insulators, as it is well known that in alternating current transmission lines, and considering the capacity with regard to the ground, the insulation of a live line is inferior to the insulation of a dead line. According to M. Friese, the phenomenon of discharge at the rim of the lower shell is due to the fact that when the surface is wet, the upper shell proves to be a conductor, and the water running along the rim is at the same potential as the line, and thereby is attracted to the pin, which is at another potential. It has also been remarked that the wind blowing horizontally helps said attraction by driving the water horizontally; cases have been observed that creeping sparks occur at a lower voltage for a wind driven rain dropping horizontally, than for rain falling vertically. Fig. 259 illustrates a protective device for transmission line insulators. The Semenza type of insulator is notable on account of its umbrella like top shell, the object of which is to prevent the rain from having any effect; the transmission line is also fastened close to the foot of the insulator, thereby reducing the moment of rupture to a minimum. Such insulators have been tested at 110,000 volts under a rain of 47 inches depth in one hour. The advantages of this kind of insulator consist in that part of transmission line in contact with the insulator is not exposed to the rain, on account of its umbrella like top shell.

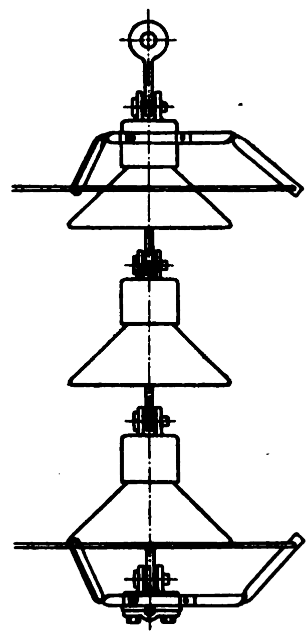
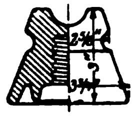
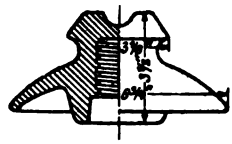


Fig. 259.
Nicholson Arcing Ring.

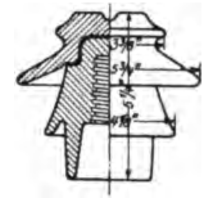
Mr. A. S. Watts, in his interesting study of insulators, has come to the conclusion that a larger insulator does not possess proportionately increased efficiency. If application could be made of the relation of increased dimension with increase in the electro-motive force of the transmission line, all insulators would possess the same relative proportions. It is clear however, that the mechanical considerations must be taken into account for low voltages, that manufacturing difficulties do suggest changes in the larger sizes, and finally, that the fundamental factor, that is to say the cost of each insulating unit determines limit of weight per insulator.



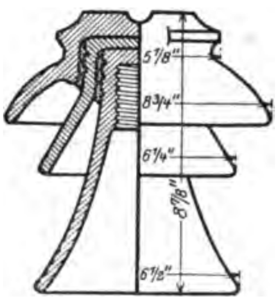
a.
6600 volts. 1 lb.



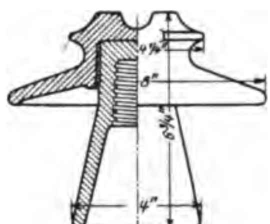
b.
13000 volts. $2^1/_2$ lbs.



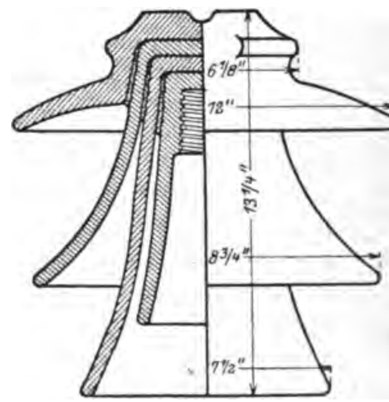
c.
18000 volts. 3 lbs.



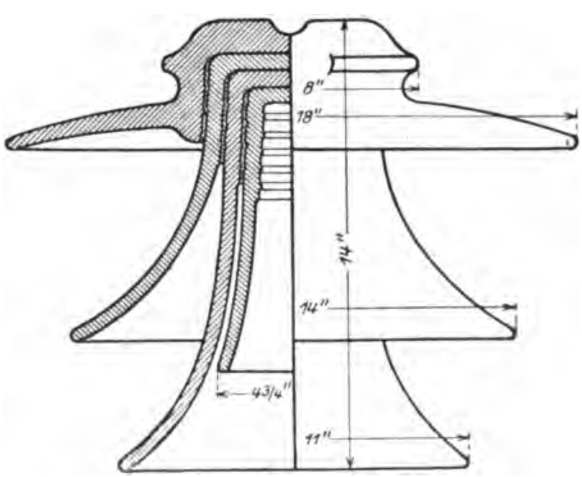
e.
40000 volts. $8^1/_8$ lbs.



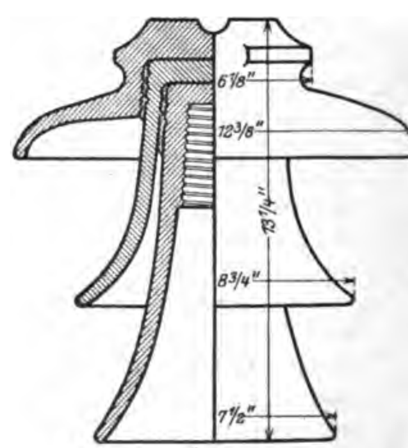
d.
27000 volts. $4^1/_2$ lbs.



f.
50000 volts. 21 lbs.

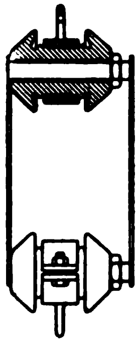


g.
70000 volts. $42^1/_2$ lbs.

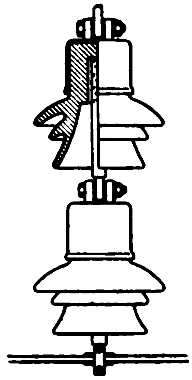


h.
60000 volts. $22^3/_4$ lbs.

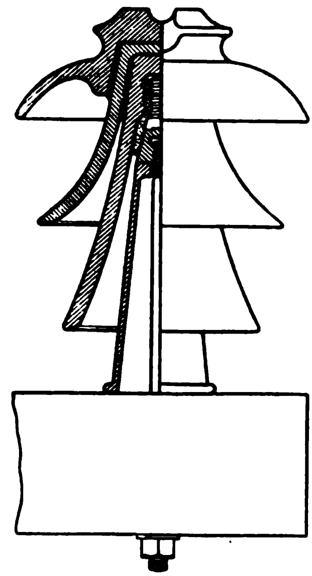
Fig. 260.



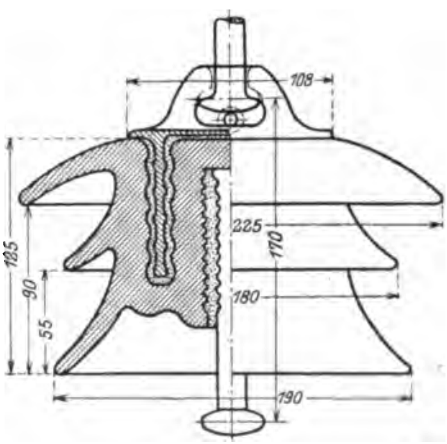
a.
Brown Bovery
Insulator.



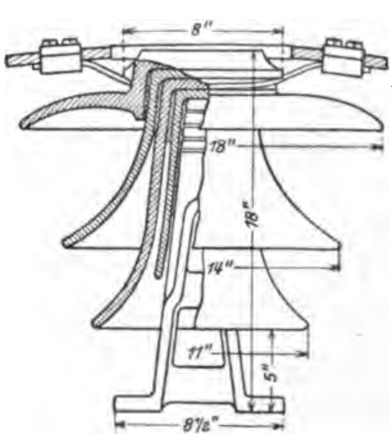
b.
Porzellanfabrik
Insulator.



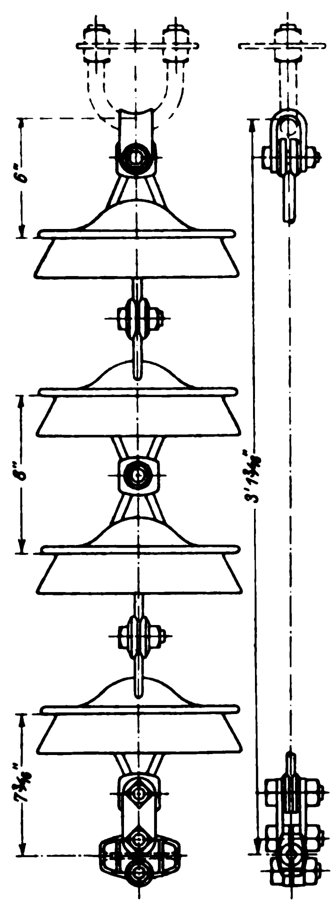
e.
50000-volt Insulator, Southern Power Company.



c.
Suspension-Insulator Unit.



d.
Insulator for 60000-volt used on the
Kern River Transmission System.



f.
Suspension-Insulator for very High Tensions.

Fig. 261.

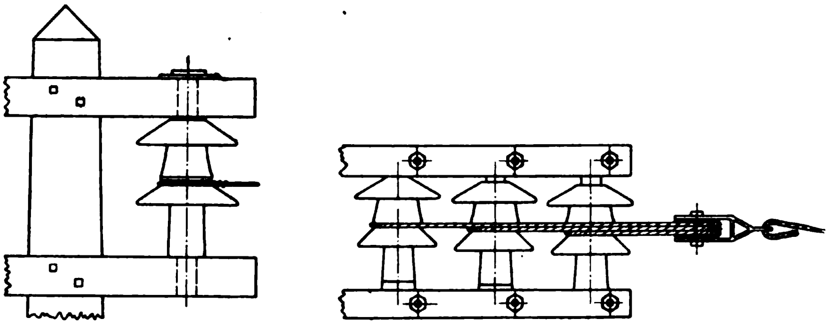


Fig. 262. Application of Cooke Strain Insulators.

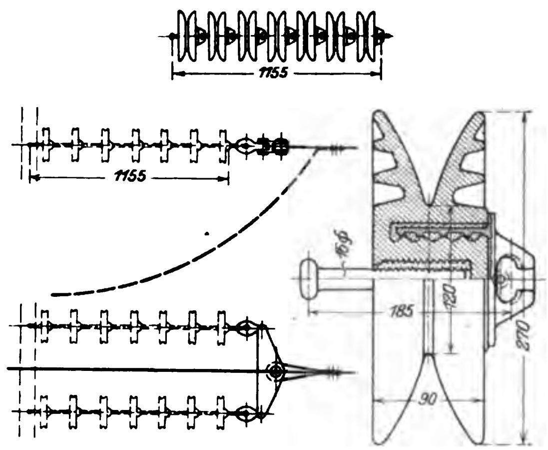


Fig. 263. Strain Insulator, German Type, 110,000 volt Transmission.

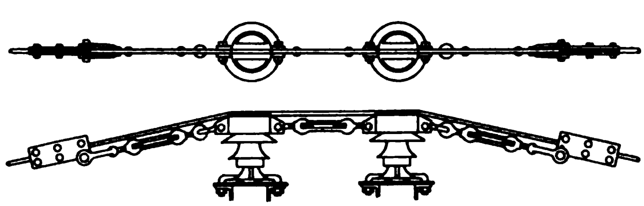


Fig. 264. Diagram of Anchor Insulators.

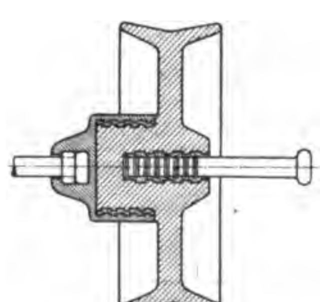


Fig. 265. Strain Insulator.
German Type.

M. Watts classifies the insulators into two categories:

a) those constructed with the idea that they shall possess enough dry surface at all times to provide the necessary insulation; and

b) those constructed upon the principle that porcelain surfaces, even when damp, possess a certain insulating value, and if enough distance is provided the required insulation is secured.

It would appear that a pin type insulator, without petticoats, but containing surface corrugations, is the most practical solution of the insulator problem. According to M. Watts, the weight should vary at a rate only slightly greater than the change in electro-motive force. The advantages in this suggested type are the following:

1.—It is not so heavy as the ordinary type of high voltage insulator;

2.—It can be handled with much less danger of breakage in construction;

3.—In case of lightning, it has no extending shells to be broken by the shock; and

4.—In the case of rain, its exposed surface is low as compared with most high voltage insulators.

A number of different types of insulators are shown in figs. 260 and 261. Strain Insulators which are placed at the beginning and end of transmission lines and sharp bends are illustrated in figs. 262—265.

152. Guard Wires on Transmission Lines.

Disturbances which may arise in transmission lines, such as are due to atmospheric conditions, are best taken care of by placing a ground along and above the line itself. This is accomplished by placing a guard wire with taps to the ground, preferably every two or three poles. On metallic poles, the ground wires are generally connected to the structure, in order to discharge to ground with the least effect upon the system.

Dr. Steinmetz says: "To protect a transmission line by overhead ground wires, however, it is essential that the line be within the protective zone of the wires, that is, within the space enclosed by an angle of 45 degrees or preferably 60 degrees from the ground wires downward."

"Barbed wire, by reason of its more rapid action, is more effective than ordinary wire against some kinds of atmospheric disturbances; for example, electro-static charges picked up from drifting rain or fog; while with other disturbances it offers no superiority over plain wire."

"The conductivity of the ground wire is of considerable importance. For the purpose of bringing ground or zero potential up to a point above the transmission line, and thereby lowering the electro-static potential of the space in which the transmission line is located, the size of the ground wire is obviously immaterial but high conductivity of the ground wire is of importance in protecting the transmission line from the inductive effects (electro-magnetic or electro-static) of oscillating, or sudden, atmospheric

discharges, such as lightning flashes; and also in protecting the stations in case of a direct stroke of lightning reaching the line, by its dampening effect as a grounded secondary conductor."

"Inasmuch however, as a system of overhead ground wires cannot be a complete enclosing shell of perfect conductivity, its protective effect, however great, cannot be quite complete, therefore protective devices have to be installed at the stations as safe-guards against the entrance of the lightning from the line."

The factors influencing the protection offered by guard wires are their inductance and specific ohmic resistance. Inductance creates a condition of self-induction, known as the skin effect, and therefore the specific resistance of the rim of the wire is of great importance. The width of the effective ring produced by the skin effect is affected by the frequency and the magnetic permeability of the material used. The galvanized guard wires which are often used are very effective, as the thin coating of zinc offers the qualities of a non-magnetic shell. As stated before, grounds should be located at close intervals; they give the best protection against atmospheric discharges and have a tendency to dampen high frequency surges in the transmission line itself.

153. Wooden Poles. Mechanical Aspects.

Transmission lines running through, or in close proximity to, a forest district, may be carried on wooden poles, because of their cheapness as far as first cost is concerned. Compared with the other types, as their life is short, and as the pole line must be as permanent as possible, steel or concrete is to be preferred when possible. Experience shows that cedar is the best wood for poles; in the South, yellow pine is the wood generally employed, whereas in the middle West, the so-called Norway pine is chiefly used. In general, it may be said that the most durable timber for poles is furnished by coniferous woods from poor soils and dense forests.

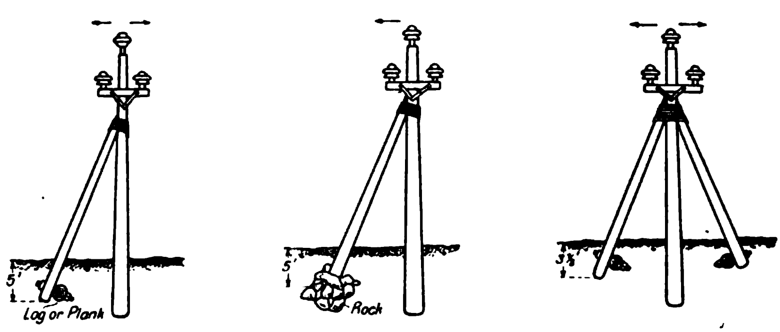


Fig. 266.

The most common sizes in use are 30 to 45 ft. in length as the high tension conductors should not be less than 20 ft. above the ground.

The standard spacings are shown in table LXII. The depth of setting varies with the character of the soil, table LXIII representing what may be

called the standard. In any case, the hole should be large enough to leave 6 inches marginal space, so that tamping may be done efficiently.

Table LXII.

No. of Conductors	Size of Conductor	Dia. of pole top inches	Pole spacing ft.	No. of poles per mile
3	No. 6—1	7	150	35
6	„ 6—1	7	150	35
3	„ 0—0000	8	125	42
6	„ 0—0000	8	110	48
3	250,000 c. m.	9	100	53
6	250,000 c. m.	10	80	66

Table LXIII.

Length of pole feet	Depth in ground feet	Length of pole feet	Depth in ground feet
25	5.0	55	7.5
30	5.5	60	8.0
35	5.5	65	8.5
40	6.0	70	9.0
45	6.5	75	9.5
50	7.0	80	10.0

If poles are set in solid rock, the setting is to be 2 ft. less than that shown in table LXIII, and in this case also, a margin should be left in order that pieces of rock may be wedged securely around the pole. When marshy land is encountered, the most reliable way to secure a pole consists in bolting cross timbers. In many cases it will be found practical to build a rigid concrete foundation. Guying of poles should be done before stringing the conductors, and when guying is found to be inconvenient in some cases, bracing is resorted to. (Fig. 266).

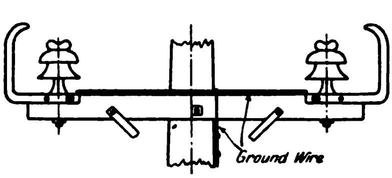


Fig. 267.

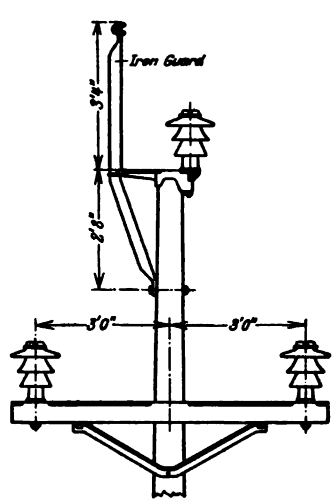


Fig. 268.

Cross-arms are generally made of

a) Long leaf yellow pine,
b) Oregon or Washington fir, or
c) Cedar, Cypress, White Pine.

Table LXIV. *Single three-phase transmission, fig. 271. No ground wire.*

Line voltage	Length of arm, ft.	A inches	B	C	Spacing of Conductor
2300—6600 incl.	3	4	1′—2″	2′—6″	2′—4″
10,000—20,000 „	4	4	1′—8″	3′—0″	3′—4″
20,000—30,000 „	5	6	2′—0″	3′—6″	4′—0″
30,000—40,000 „	6	6	2′—6″	4′—6″	5′—0″
40,000—50,000[1] „	6	6	2′—6″	4′—6″	5′—0″
50,000—60,000[1] „	7	6	3′—0″	5′—0″	6′—0″

[1] Steel towers are generally used for these voltages.

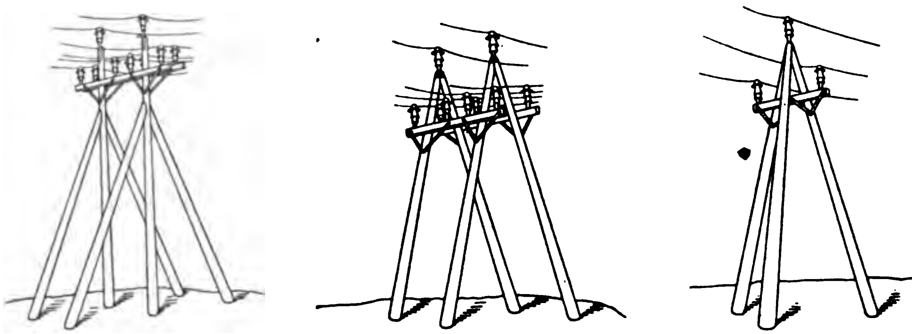


Fig. 269.

Table LXV.

Twin three-phase transmission, fig. 272. With or without ground wire.

In the figure, F is without ground wire, G is with ground wire.

Line voltage	Length of upper arm, ft.	Length of lower arm, ft.	A	B	C	D	E	F	G	H
2300—6600 incl.	7	5	4″	1′—6″	1′—8″	4″	2′—2″	8″	4′—0″	2′—0″
10,000—20,000 „	8	5	4″	2′—0″	1′—8″	4″	2′—2″	8″	5′—0″	2′—0″
20,000—30,000 „	10	6	6″	2′—6″	2′—0″	6″	2′—6″	10″	5′—6″	2′—6″
30,000—40,000 „	12	7	6″	3′—0″	2′—6″	6″	3′—0″	10″	5′—6″	3′—0″

The length of cross arms may be determined from tables[1] LXIV and LXV and they should be protected by strap iron guards (fig. 267 and 268) for the purpose of preventing their burning if the live conductor should fall upon them.

[1] General Electric Co.

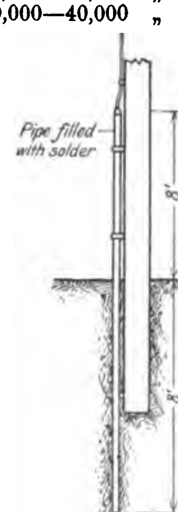


Fig. 270. Chicago Edison. Method of Grounding

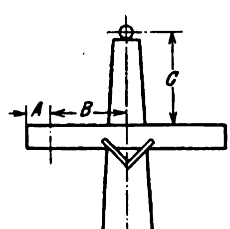


Fig. 271.

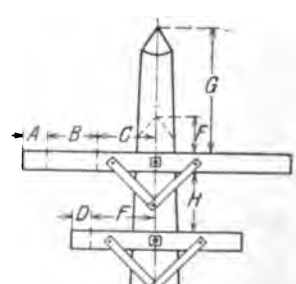


Fig. 272.

Frames for heavy strain terminals are shown in fig. 269.

The grounding of a pole must be made with special regard to permanent moisture; the method of grounding of the Chicago Edison being illustrated in fig. 270.

The stresses in a wooden pole of uniform cross section may be determined by the following formula

$$M = \frac{\pi R^3 S}{4 H}$$

in which

M = bending moment,
R = radius of section at ground level,
S = strength of wood per sq. inch, and
H = height above ground of applied force.

154. Wooden Pole Preservation.

A wooden pole transmission line is undoubtedly the cheapest in first cost, but not so far as maintenance and depreciation are concerned. Steel tower transmission lines may last about 30 years whereas wooden poles used instead last only 12 years as an average. The life of such a pole varies with the different kinds and character of wood. Preservative methods are therefore resorted to in order to prolong their usefulness. It has nearly always been thought that the preservative medium should contain a germicidal or antiseptic element, but it has been observed lately that the so-called house fungus, being the most powerful destructive agent of wooden poles, can be prevented from thriving by the elimination of air and moisture, the life of poles attacked by these parasitic fungi being limited to from two to fourteen years. The new conception of preservation is the crude oil treatment. This method consists in first seasoning the wood by the direct application of steam, admitted in the treating cylinder into which the poles have been placed in large number, the temperature being maintained to about 250° Fahr. for from 6 to 20 hours. Then the steam is allowed to escape, and a vacuum is created in the cylinder, forcing the sap of the poles to flow outwards. After this has been completely removed, crude oil is admitted into the tank, which, when filled, is subjected to hydrostatic pressure until the lumber has absorbed the specified amount of oil. The success of the treatment depends upon the thorough impregnation of the sap wood with all

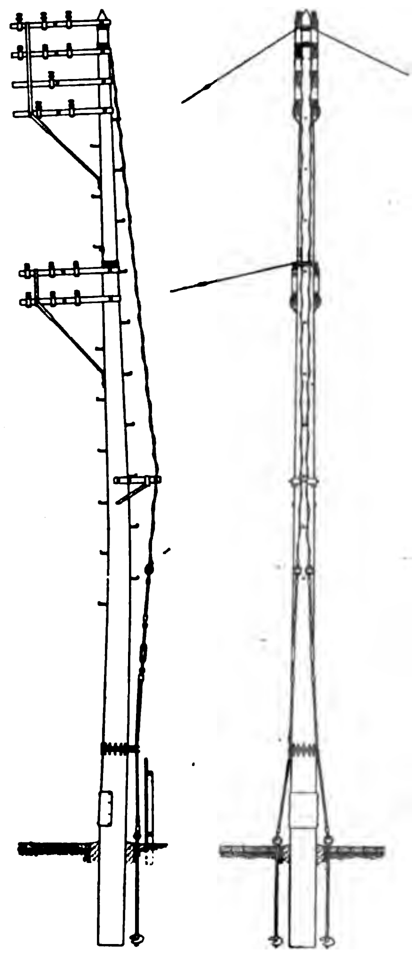


Fig. 273.
Method of Strengthening Corner Pole.

the oil that it is possible to force into it. There are also other liquids used to prolong the life of wooden poles, the method of application being about the same as the crude oil process. Copper sulphate and zinc chloride have been used to some extent but their success has been limited. Bichloride of mercury is a much more powerful agent, and in some cases has given even better results than those obtained by the use of creosote oils. Sodium fluoride and zinc fluoride have also been used successfully. Impregnation with tar is the best preventative from the fungi.

The average life of poles for different impregnating materials has been given by the Imperial German Postal Department, to be as follows:

Zinc chloride	12 years
Copper sulphate (blue vitriol)	14 „
Corrosive sublimate	17 „
Tar oil	22 „

These figures show that impregnation with tar oil is most advantageous.

White cedar is the best material for poles on account of its comparative durability in contact with the soil. Such poles are lasting from 18 to 20 years in ordinary soil without treatment. A 20 ft. pole is set from 3.5 to 4 feet in the ground, and the depth of the setting varies with the length of the pole: a 5 feet setting for 35 ft. poles and a 7.5 ft. setting for 80 ft. poles.

Where the soil is subject to sudden alternations of dampness and dryness, the decay is occasioned at a greater rate. If cut in winter and seasoned immediately, wooden poles last longer than if they are cut in summer.

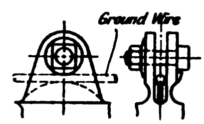
155. Steel Towers. Flexible Towers.

As is the case with most modern generating plants, the electric energy is transported over considerable distances, a problem that is solved by the conveyance of such energy under high voltages. The security of the transmission line is therefore the most vital part of the system, and great care must be exercised in order to eliminate danger to life, minimize fire risks, and prevent the possibility of shut downs due to mechanical failure of the concerned parts. In all cases it must be considered that any important transmission line is to be built with an idea of permanency. The mechanical part of a transmission line then consists in the economical design of the tower or supports, with the idea of maximum security.

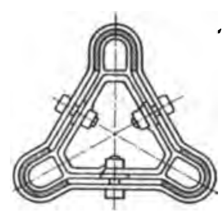
The use of steel towers is resorted to for several reasons: the rise in the price of wooden poles, the reduction in the price of structural steel; steel towers may be built to greater heights than would be reached by wooden poles, with an increase in security, increase in span, and better economies.

The design of steel towers for transmission lines presents many difficulties, or rather, complications. As their chief advantage lies in their use in long spans, yet, it is obvious that long spans introduce other difficulties consisting in possible breaks due to emergency loads caused by wind, sleet, temperature, etc.; but then also longer spans have their advantages ;-a reduction in the number of insulators and the resultant decrease in chances of a break-down. Economical towers are best secured by reducing the height as much as conditions will permit. In fact, the cost of a steel tower, accord-

ing to M. Scholes, varies about as the cube of the height for a given design, and as the questions of sag and strength of conductors are of prime importance and determine the height of towers, the conductors must be drawn to their maximum possible tension, a result that is attained by the use of hard drawn copper and aluminum of high elastic limit. Naturally, such high working stresses exact new precautions in respect to the design of joint cross arms and insulators. Long spans allow vibrations of the conductor, necessitating the use of flexible insulators, a matter that is arranged by the use of a suspension insulator. This type offers many advantages over the pin type of insulator. The latter is used for tensions reaching 60,000 volts, but practice has shown that, above this voltage, the cost of pin insulators increases as the cube of increase in voltage, and further as they increase in length they impose torsional strains, thereby affecting the strength of cross-arms. The suspension type of insulator is flexible, and although it requires a tower higher than the one that would be required for the corresponding pin insulator, it eliminates the possibility of torsion on the arms and head of the tower. The early observation that the best transmission line is the one which possesses greatest elasticity, resulted in the design of flexible towers. As a matter of fact, rigid towers necessitate the use of many light parts liable to deteriorate rapidly by corrosion. A less complicated design, using heavier parts, does not only last longer but also costs less to maintain. Italian engineers were the first to recognize these advantages, and the result of these observations were embodied in the Semenza type of flexible tower. One of its advantages consists in the fact that in case of wire breakage, the impact is absorbed into a flexible system, whereas in the case of rigid towers, the impact met by a rigid body results in damages from recoil. A flexible



Detail at "A"



Section at Collar and Spreader.

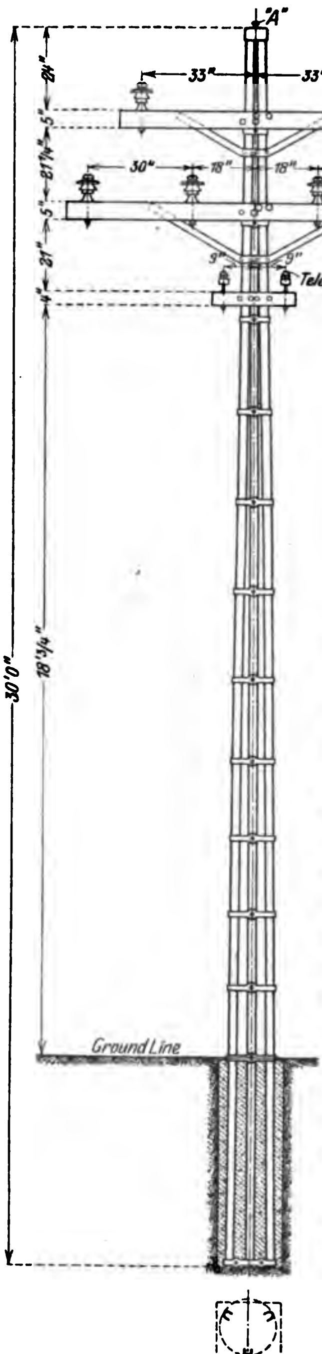


Fig. 274. 30 ft. Tripartite Steel Transmission Pole. Franklin Steel Co.

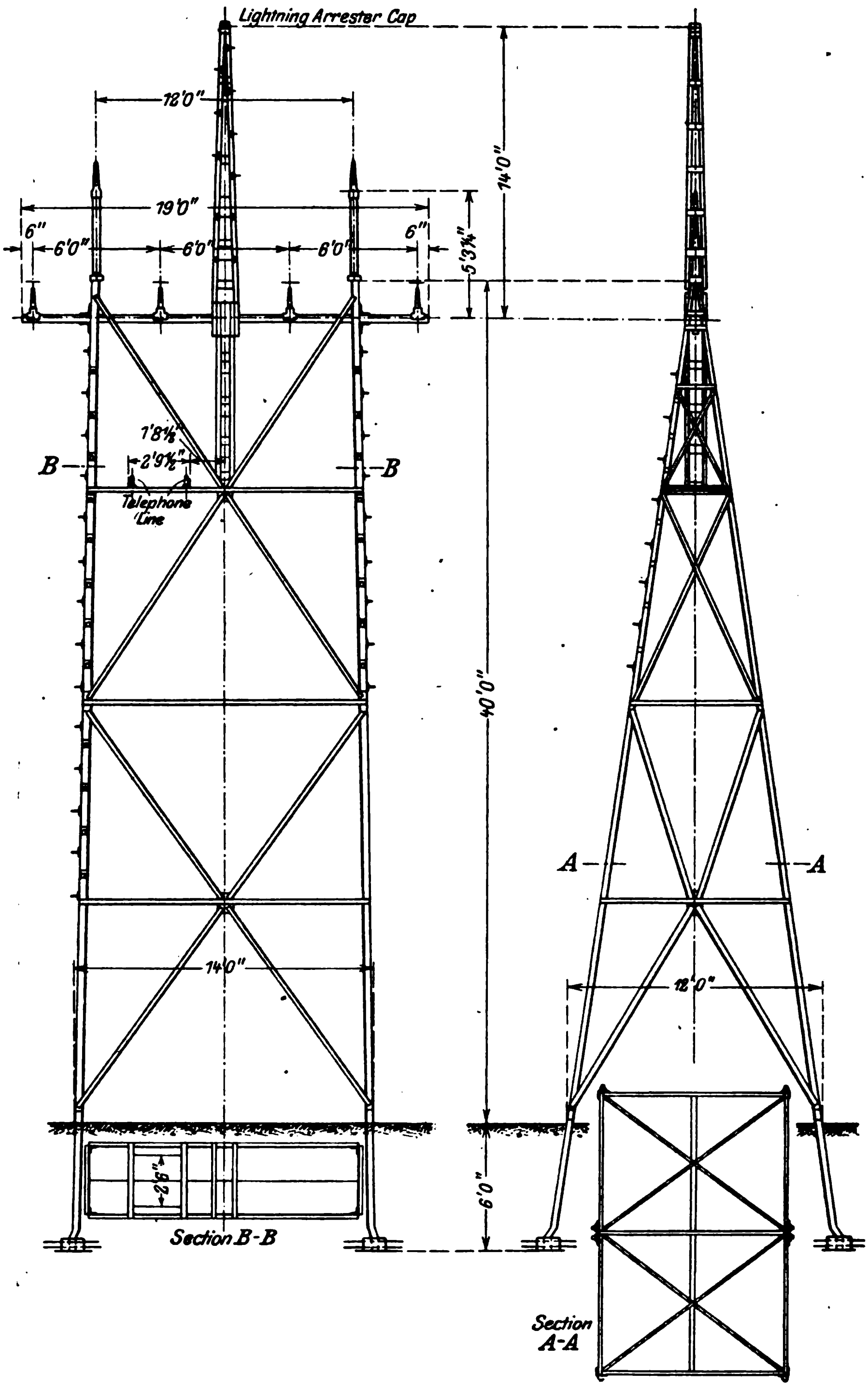


Fig. 275. 40-foot Transmission Tower, Necaxa, Mexico. Similar Type of Tower is used in the 80,000-volt Transmission Line between Rio das Lages and Rio de Janeiro of the Rio de Janeiro Tramway, Light and Power Company, Brazil.

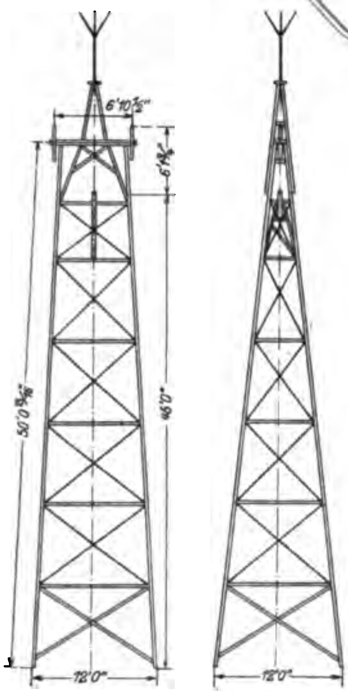
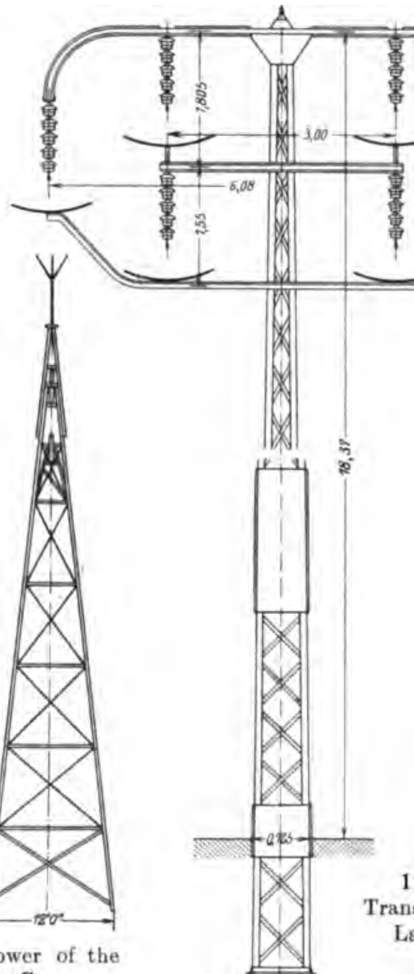
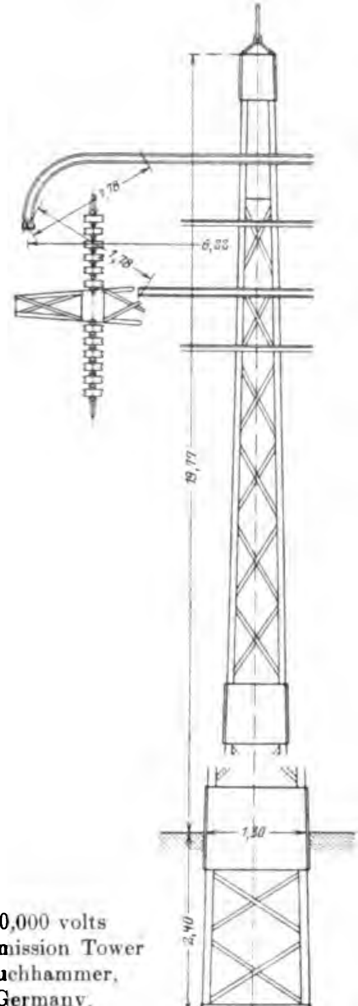


Fig. 276. Type of Tower of the Arnherst Power Co.





110,000 volts Transmission Tower Lauchhammer, Germany.

Fig. 277. Intermediate Tower.

Fig. 278. Straight Tower.

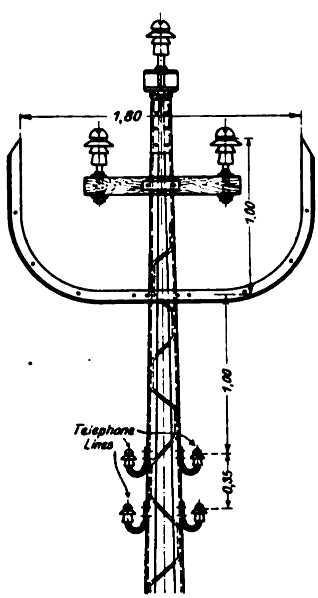


Fig. 279. Type of 35,000-volt Tower with Brackets for Guard, Heimbach Plant, Germany.

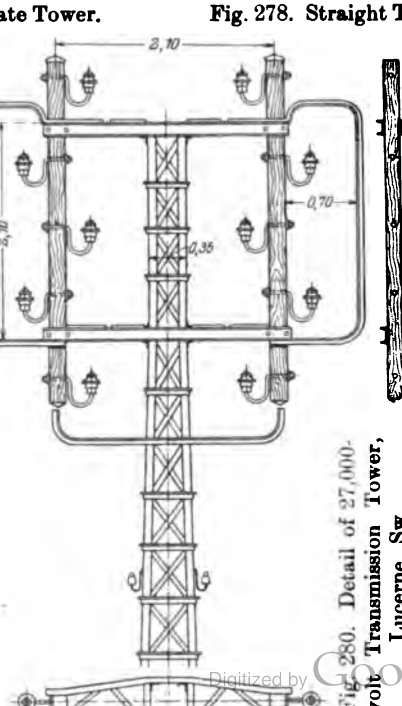


Fig. 280. Detail of 27,000-volt Transmission Tower, Lucerne, Sw.

tower does not necessarily mean a tower that is flexible in all directions: it is flexible only in the direction of the line and rigid sideways and vertically. Such structures consist of two upright members, cross-braced and supplied with cross-arms strong enough to transmit the maximum wire stresses to the tower itself. The best method of transmission line design consists in placing at regular intervals along a flexible tower line, a double guyed structure,

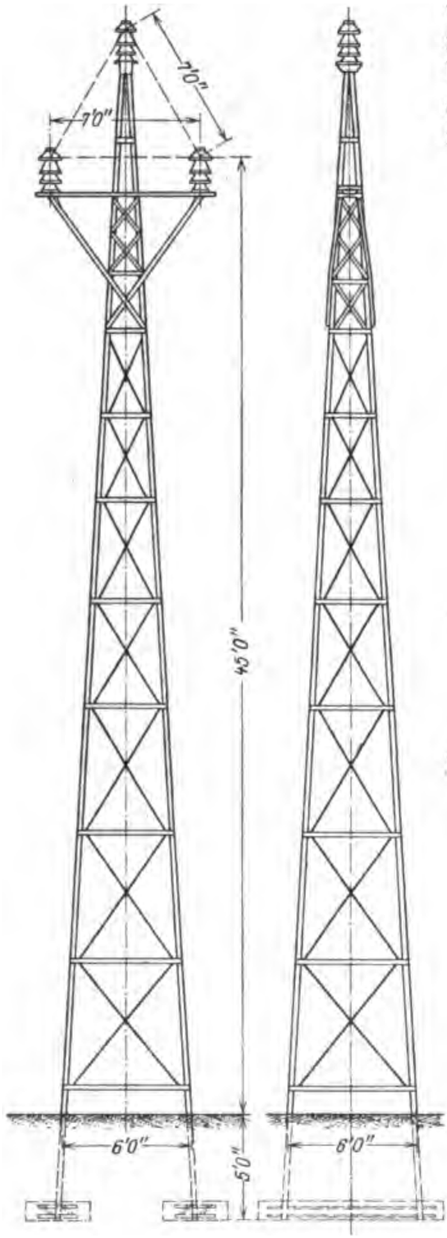


Fig. 281. Detail of 45-Foot Angle Iron Tower at Syracuse, N. Y.

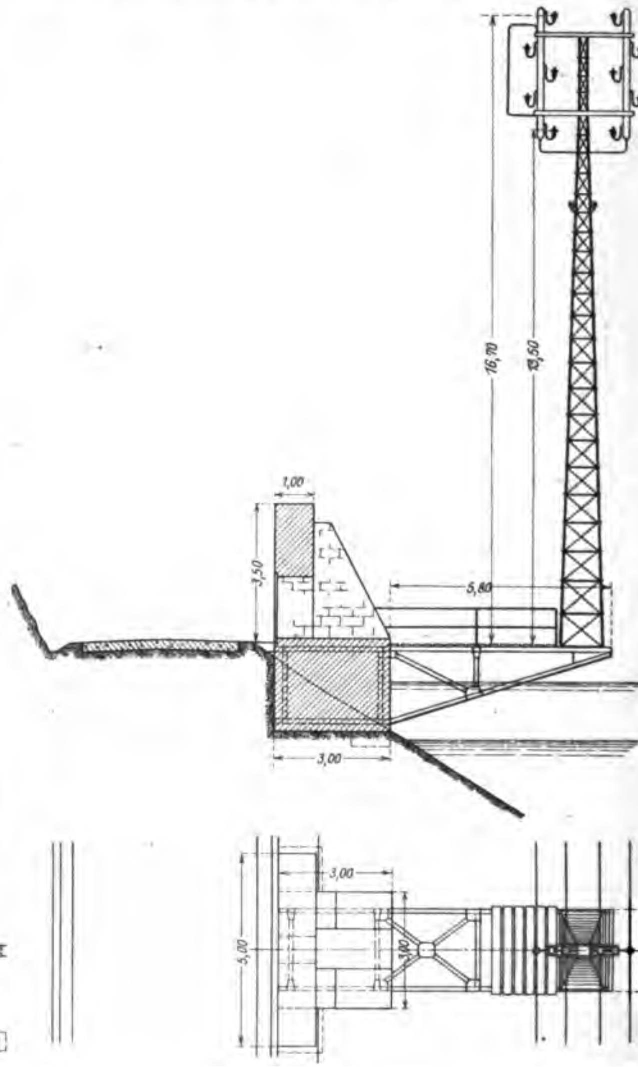


Fig. 282. Cantilever Construction of 27,000-volt Transmission Tower, Obermatt-Lucerne, Switzerland.

or a dead-end tower. These are preferably placed approximately every mile, and should be designed strong enough to stand any strain likely to occur on the line. In cases of rupture the stresses will be dampened to some extent by the flexible system and the excess will be in a degree transmitted to the dead-end towers.

Steel poles are also used for light transmission lines instead of wooden poles, allowing spans of 300 feet. The advantages of steel poles are similar to those of flexible towers, but precautions should be taken in their foundations. They should be concreted into the ground, a method that is recommendable for towers as well; the structure is thereby greatly strengthened, and corrosion reduced to a minimum.

A peculiar and very successful pole, known as the "Tripartite" pole, fig. 274, has been recently developed, is of a very simple construction, and radically different in design from any pole hitherto employed. These poles are set in concrete to a depth equal to one tenth the entire length of the pole, and this anchorage has been found sufficient by repeated tests, to withstand a horizontal strain equal to the strength of the pole itself. They have been

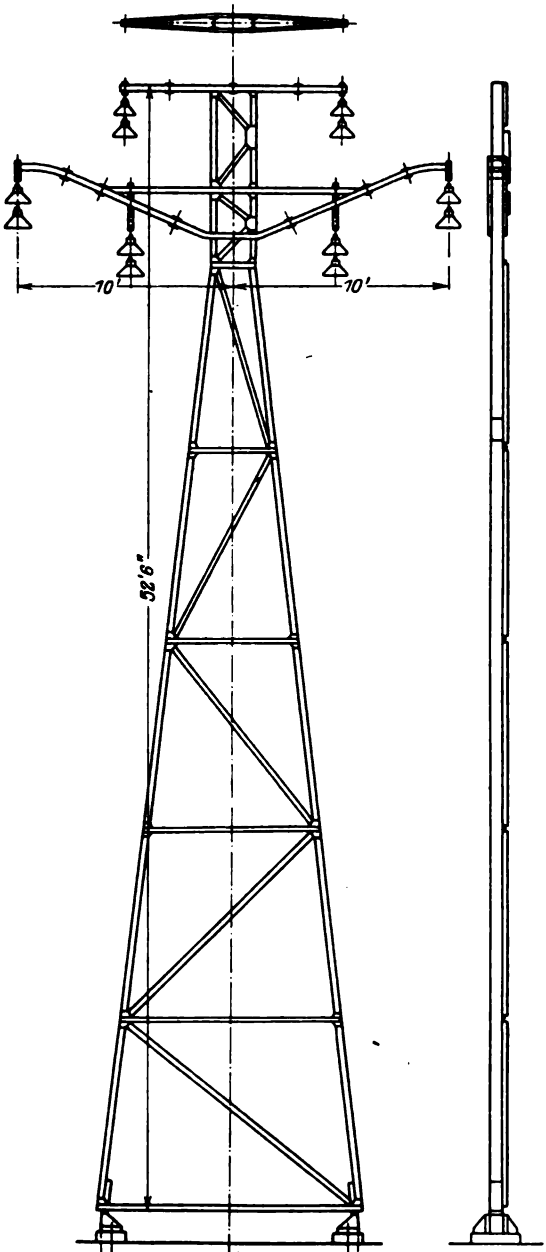


Fig. 283. Royal Trollhattan Canal Tower.

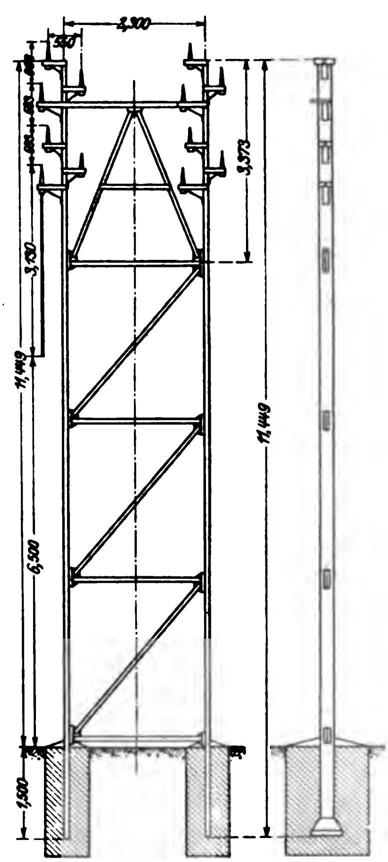


Fig. 284. Two-Legged 37-Foot Transmission Tower, Used in Switzerland and Italy.

used by the U. S. Government for a 40,000 volt transmission line in Arizona, the height of the pole extending 55 feet above the ground.

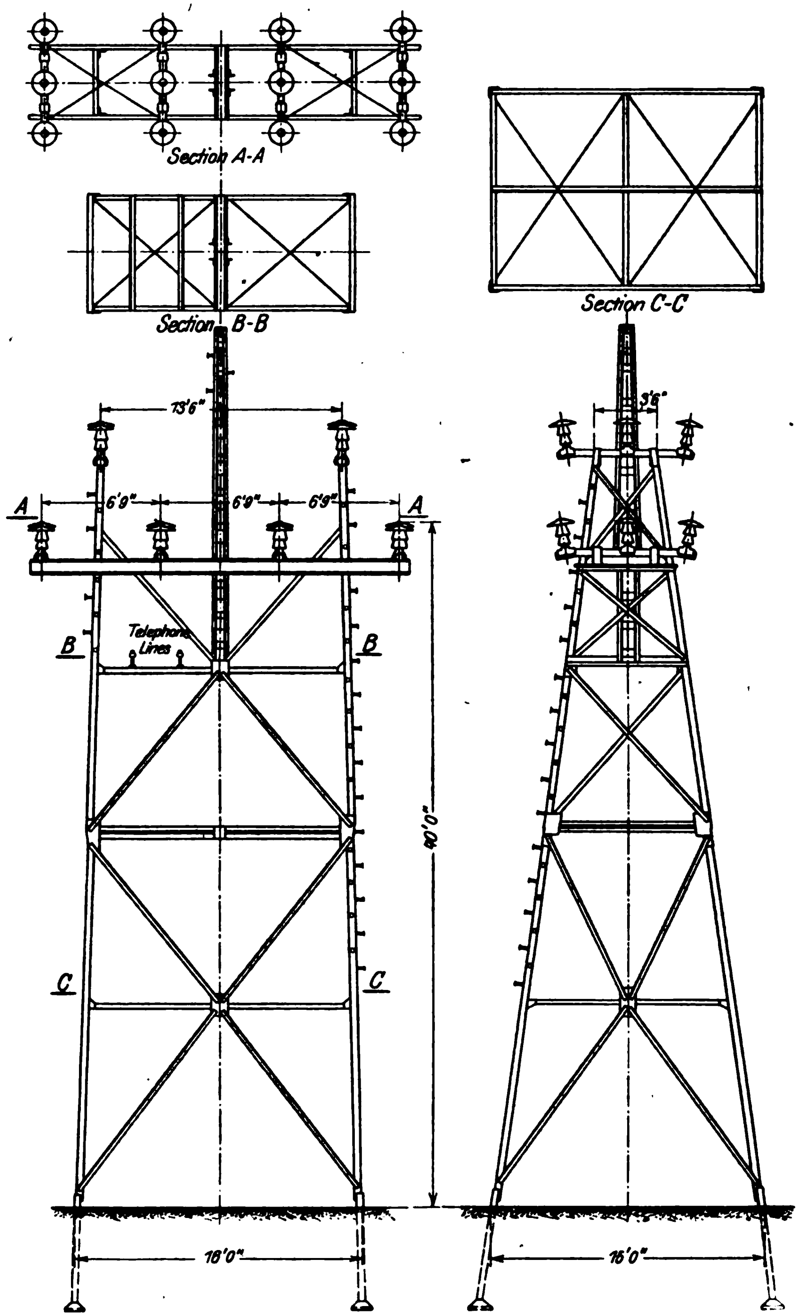


Fig. 285. Two Circuit Tower, "Duluth" Type, Riter-Conley Mfg. Co.

The design of transmission towers, is a work incumbent upon structural engineers, and the attention of those interested is drawn to the illustrations, figs. 275 to 286, in which there will be found considerable information regarding general design and detail of construction.

156. Conditions governing Tower Design.

In order to properly design towers for high tension transmission lines the following conditions must be considered:

Location of the line,
Length of the transmission line,
Proposed voltage,
Size and material of conductors,
Number of circuits,
Number of conductors to each circuit,
Type of insulators,
Minimum clearance between lowest sag of wire and ground,
Minimum and maximum temperature,
Minimum wind velocity,
Number, size, and material of telephone wires,
Number size and material of ground wires.

157. Durability of Steel Towers.

With the rapid increase in the use of steel towers for transmission lines, and the experienced rapid corrosion due to climatic and electrolytic causes, the problem of preserving such structures has become a very serious one.

Steel towers in general, are attacked at the base and more especially at the exposed joints, the rivets being very susceptible to rapid corrosion. To preserve these in good condition, they must be thoroughly galvanized and well protected by paint. It is needless to remark that an "ideal" tower should unite the following qualities:

Indefinite duration,
Fireproofness,
Reduced cost of maintenance,
Reasonable lightness,
Flexibility within safe limits, and a
Relatively low cost.

"American engineers", says the Electrical World, "have been in the habit of making extreme and needless requirements of stiffness in the towers, some going so far as to assume the simultaneous breaking of all the wires supported when under the heaviest possible load (an accident which has been practically unheard of in the history of pole lines), as the criterion of suitable strength. On top of such absurd requirements for load, they have demanded stiffness instead of encouraging the flexibility, which relieves the strains due to the line by flexure, within the elastic limit, and have still further insisted on cheapness as a final virtue. Now a structure at once needlessly stiff, needlessly strong and very cheap is not easy to design, and in attempting economy of material to meet the last requirements the tendency has been toward somewhat intricate tower structures made of material altogether too thin for sound construction. It is this light construction, with innumerable joints and rivets, that is likely to lead to trouble from corrosion.

"A line that is to be a permanent investment is worth putting up well,

and should be designed as an engineering structure with permanence in view; otherwise the economy of steel tower construction is likely to be forfeited through unexpected brevity of life. A steel tower line designed to stand any practical strain, to yield and relieve abnormal strains, as in the Semenza type used in Italy and now being introduced here, and to last for a long term of years, can easily be secured by intelligent engineering. The next thing is so to care for it that it will not go out of service from local corrosion, which is likely to be the fate of a very large proportion of steel towers now in use. It is not only necessary that the steel be well galvanized, but that it should be well protected by paint, especially for the purpose of keeping the weather out of riveted joints, and should be inspected and repainted as conscientiously as if the line were a wooden one to be watched for incipient rotting. With such precautions carried out there is every reason to believe that a steel pole line will give a useful life of a half century or more. If, on the other hand, a line is built of extremely light towers designed to meet the fanciful requirements of stiffness while economizing material to the last possible degree they are likely to be of material far too light to permit even moderate corrosion without disastrous results; and such a line, put up with implicit trust, with indifferent galvanizing, and seldom painted and inspected is more than likely to suddenly fail before it has been in service many years, and to become a continual source of anxiety and annoyance, particularly since thin sections of steel may be dangerously corroded without showing the injury except on very careful inspection."

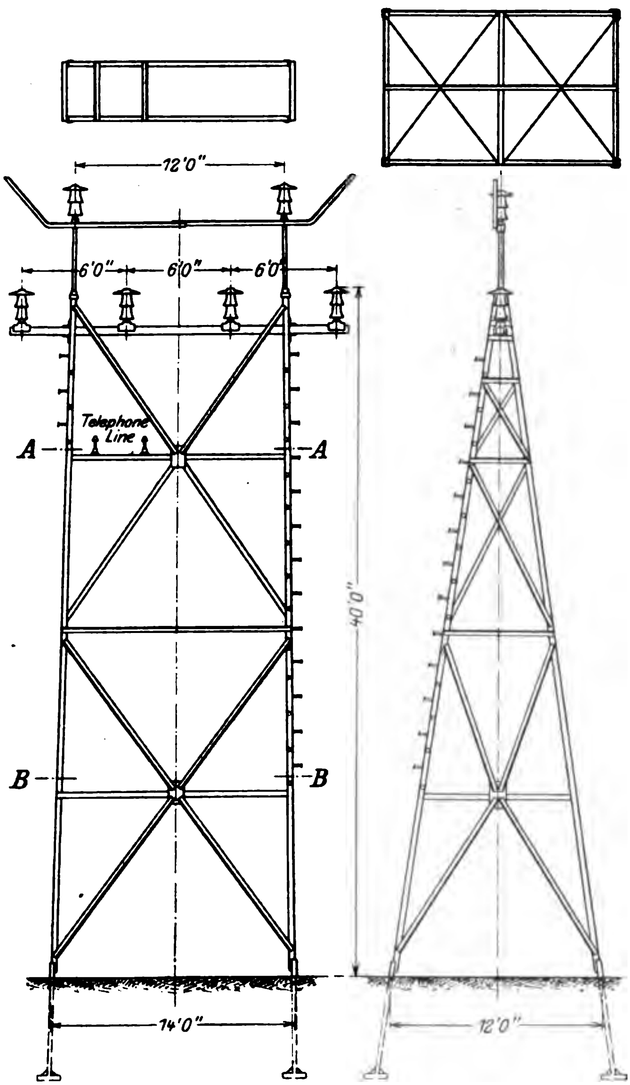


Fig. 286. Two circuit Tower, "Juniata" Type, Riter Conley Mfg. Co.

158. Standard Specifications for Galvanized Steel Poles[1].

Each pole shall be constructed in accordance with the general drawings giving the following dimensions. The height of pole from top to bottom, including the stub ends, shall be 82′ – 3″.

The stub ends shall be designed so that the same shall be embedded in the ground 6 ft. The pole shall have three (3) cross-arms; the bottom of the lowest cross-arm shall be 51′ – 2″ above the ground; the bottom of the second cross-arm shall be 10 ft. above the lowest cross-arm; the third cross-arm shall be 10 ft. above the second cross-arm; the extreme top of the pole shall be 5′ – 1″ above the bottom of the highest cross-arm. The two top cross-arms shall be of such length that the feed wires shall be 17′ – 1″ from each other in horizontal plane. The lowest cross-arm shall be of such length that the feed wires shall be 18′ – 1″ apart, measured in a horizontal plane. The corner legs of the tower shall measure 17 feet apart at the ground level, on all four (4) sides.

The pole shall be capable of standing the following tests, when tested in a vertical position on an immovable foundation: a simultaneous horizontal pull of 1000 lbs. at the top and 2500 lbs. at each of the three (3) cross-arms; or in other words a total strain of 8500 lbs. The cross-arms shall carry at each suspension point a weight of 1000 lbs. Any one of the cross-arms to carry at one of its ends a horizontal pull along the line of 3000 lbs. The poles shall carry the above loads without serious permanent deflection of any of the members. All tests, inspection and acceptance of the finished poles must be made at the point of manufacture before shipment.

The pole shall consist of stub ends, pole proper with all of its bracing, steel cross-arms, and the necessary bolts for erection in the field. When the suspended form of insulators is used, a plate with a hole shall be provided at the end of the cross-arm for hanging insulators. When pin insulators are used, holes shall be provided in the top flange of the cross-arm for securing the same. No insulators, nor wires, nor other material is to be included. The pole is to be constructed entirely of open hearth medium steel of the following specifications:

Ultimate strength 60,000 to 70,000 pounds per square inch;

Elastic limit not less than one half the ultimate;

$$\text{Percentage of elongation} = \frac{1{,}400{,}000}{\text{ultimate strength}}.$$

Bending test, 180 degrees to a diameter equal to the thickness of piece tested, without fracture on outside of bent portion. For each decrease of 1/16″ in thickness below 5/16″, a deduction of $2\frac{1}{2}$ per cent shall be made from the specified elongation. In rounds of 5/8″ or less in diameter, the elongation shall be measured in a length equal to eight (8) times the diameter of the section tested.

All tests and inspection of steel shall be made at the rolling mills prior to shipment.

[1] Milliken Bros., New York.

The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece cut from the finished material, in accordance with the American Manufacturers' standards.

Two test pieces shall be taken from each melt or blow of finished material, one for tension and one for bending, but in case either test develops flaws, or the tensile piece breaks outside of the middle third of its gauged length, it may be discarded and another test piece substituted therefore.

All finished bars shall be free from injurious seams, flaws or craks, and have a workmanlike finish.

All pieces must be finished straight and true in the shops must be carefully cut to length, and punched so that the pole can be put together without unnecessarily straining or distorting the material (except pieces that are specially intended to be bent in the field). The punched holes must be so accurately spaced that the pole will go together without the necessity of drifting and distorting the material. The punched holes shall not exceed the diameter of the bolt by more than $\frac{1}{32}$ of an inch.

After all of the shop work has been finished, all structural parts of the pole must be thoroughly galvanized by using the hot process. Each piece galvanized shall be dipped in the bath at one operation, that is, it will not be allowed to dip a piece one end at a time.

The galvanizing shall consist of a coating of metallic zinc, evenly and uniformly applied, so as to firmly adhere to the surface of the metal. All galvanizing (except as hereinafter provided) shall be done by the hot process, after all punching, cutting and other machine work are completed. All galvanized work shall be subjected to the following test:

A solution of sulphate of copper shall be made, using commercial copper sulphate crystals in water having a specific gravity of 1.185 at seventy (70) degrees Fahrenheit. The testing solution shall have a maximum temperature not exceeding seventy (70) degrees Fahrenheit or a minimum temperature of not less than sixty (60) degrees Fahrenheit. A sample piece galvanized shall be immersed in a standard solution, as above described, for one minute, and the removed, immediately washed in water thoroughly and wiped dry. This process shall be repeated. If after the fourth immersion there shall be a copper colored deposit on the sample, or if the zinc should be removed, the lot from which the sample was taken shall be rejected.

The bolts and nuts are to be galvanized by the electric process, and are not subject to the sulphate of copper test. The work is to be carefully done, and threads are to be clean, so that the nuts can be easily turned on the bolts.

The poles are to be shipped all "knocked down", that is, in separate pieces. All members that are duplicates and form one pole, are to be wired together and the bundle tagged with the proper erection mark. The bolts are to be boxed so that one box contains all of the bolts necessary to put one pole together, with an excess of five (5) per cent.

Before acceptance and shipment the manufacturer shall, at his own expense and in the presence of the inspector appointed by the owners, put together one pole complete on the ground, to show that all of the shop work has been properly done.

159. Reinforced Concrete Poles.

Reinforced concrete poles realize about all the ideal characteristics of perfect structures; long life, fireproofness, maximum economy in construction, maintenance and repair. However, they have not been extensively used as yet, very few having been adapted for transmission line work. The pole is calculated under the same basis as a wooden pole, that is, considering it as a cantilever beam rather than a column, and if it is desired that the pole be designed with special regard to economy, it is possible to use high unit stresses for conditions that may never occur. In regard to the factors of safety, unit stresses and working stresses to be allowed in the constituent materials of a reinforced concrete pole, Messrs Coombs and Slocum[1] state that there is much latitude of judgement, as in other structural work. If the reinforcing material is concentrated in four equal areas, a rod to each corner, a square pole will be equally strong, either parallel or normal to the line, and in this case concreting operations are more easily accomplished. In solid poles, the loading produces a low compressive stress in the concrete so that a large amount of concrete area may be suppressed, and the pole made hollow. Such a structure however, offers considerable difficulty in construction , and forms are rather costly.

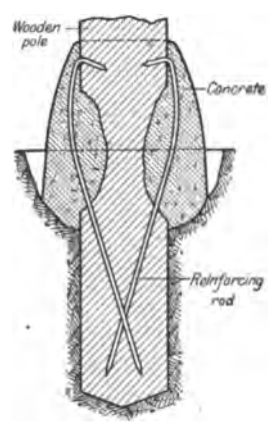


Fig. 287.

A reinforced concrete pole eliminates many inconveniences of the wooden poles, but experience shows that a combination of the two kinds gives better results, and is handled more easily. The practice therefore is to cover wooden poles with a concrete layer made to adhere by means of spiral wound rods of the mechanical bond type. The size of the wooden pole is first determined as though it is to resist by itself the combined forces due to the dead load, ice and wind loads of the transmission line.

The mixture of concrete generally employed for solid concrete poles consists of one part of Portland cement to five parts of gravel, graded from sand to pieces that will pass a $\frac{3}{4}$ inch screen. Another mixture commonly used consists of a mixture 1 : 2 : 4 Portland cement, sand, and broken stone, mixed wet, and the material carefully selected. The Marseilles Land and Water Company of Marseilles, Ills., is operating about thirty miles of 30,000 volt transmission line carried mainly on concrete poles. This line has now been about four years in operation, and the concrete poles have given most satisfactory service.

The fig. 287 illustrates the method used by the Pittsburgh Reinforcing Pole Company, for wooden poles having decayed portions at the ground line.

[1] A paper prepared by the Association of American Cement Manufacturers by R. D. Coombs and C. L. Slocum.

160. Strength of Conductors and Line Stresses.

Before calculating the sag in a conductor, the maximum stress which can be permitted in the wire must be assumed beforehand. The average modulus of elasticity for copper and aluminium wire and cable is as follows:

Copper hard drawn wire	16,000,000
Copper hard drawn cable	12,000,000
Aluminium hard drawn wire	10,000,000
Aluminium hard drawn cable	7,500,000

Calculations are generally based on the following data

	Aluminium	Copper
Elastic limit	14,000 lbs	40,000 lbs
Stress at $\frac{1}{2}$ elastic limit	1470 „	2640 „
Stress at elastic limit	2940 „	5280 „
Coefficient of expansion	0.000013	0.0000096
Variation in temperature . . .	150° F	150° F

If $D =$ deflection in feet,
$S =$ span in feet,
$W =$ resultant of weight and wind in lbs. per ft. of cable, and
$T =$ stress allowed in cable in lbs. $= 2640$ for copper;

then
$$D = \frac{WS^2}{8T}.$$

This formula gives
$$T = \frac{WS^2}{8D} \quad \text{and} \quad S = \sqrt{\frac{8DT}{W}}.$$

The tension at the points of support is a little higher than the tension at the middle of the span, its value being:
$$T_1 = \frac{WS^2}{8D} + WD.$$

The length of the cable (cold) is given by:
$$L = S + \frac{8D^2}{3S}$$
from which
$$D = \sqrt{\frac{3S(L-S)}{8}}.$$

The length of the cable without stress is determined by
$$L_0 = \frac{L}{1 + \frac{F}{E}}$$
in which

$F =$ lbs. per sq. in. permitted in cable, and
$E =$ modulus of elasticity.

The length of cable (hot) is:
$$L_H = L_0(1 + CB)$$
in which

$C =$ coefficient of expansion $= 0.0000096$ for copper; and
$B =$ maximum degrees F rise in temperature $= 150°$ F.

Table LXVI. *Stresses in wires and cables.*

Size B. & S. and cir. mils	All Materials: Cross-section sq. in.	Projection sq. in.		Hor. force lb. per ft.		Aluminium: Weight lb. per 1000 ft.	Result. Force: lb. per ft.		lb. per ft. per sq. in. cross-section		Copper: Weight lb. per 1000 ft.	Result. Force: lb. per ft.		lb. per ft. per sq. in. cross-section	
		Wire	Cable	Wire	Cable		Wire	Cable	Wire	Cable		Wire	Cable	Wire	Cable
10	0.00815	1.224		0.0893		9.54	0.0806		9.9		32	0.0948		11.63	
9	0.01028	1.368		0.0999		12.04	0.1004		9.77		39	0.107		10.4	
8	0.01297	1.542		0.1125		15.17	0.1136		8.71		50	0.1232		9.5	
7	0.01635	1.731		0.1263		19.13	0.1278		7.82		63	0.1399		8.56	
6	0.02062	1.945		0.1418		24.12	0.1436		6.96		79	0.1622		7.86	
5	0.02600	2.183		0.1592		30.40	0.162		6.24		100	0.188		7.24	
4	0.03278	2.451	2.76	0.1786	0.201	38.5	0.1827	0.2048	5.58	6.26	127	0.219	0.238	6.68	7.26
3	0.04134	2.754	3.12	0.2008	0.227	48.5	0.2065	0.2322	5.00	5.62	160	0.257	0.278	6.22	6.60
2	0.05213	3.091	3.60	0.2253	0.262	61.0	0.2332	0.2495	4.48	4.79	203	0.303	0.324	5.82	6.33
1	0.06573	3.472	3.96	0.2532	0.289	77.0	0.2638	0.2991	4.02	4.55	255	0.3585	0.411	5.46	6.26
0	0.08289	3.900	4.44	0.2845	0.324	97.1	0.3007	0.3385	3.63	4.08	322	0.434	0.460	5.24	5.55
00	0.10452	4.378	5.04	0.3191	0.367	122.4	0.342	0.387	3.27	3.70	406	0.517	0.548	4.95	5.24
000	0.13179	4.915	5.64	0.3585	0.411	154.4	0.390	0.439	2.96	3.33	513	0.626	0.657	4.75	4.98
0 000	0.16619	5.520	6.48	0.4025	0.473	194.7	0.447	0.512	2.69	3.08	645	0.760	0.800	4.57	4.82
250 000	0.1965		6.96		0.508	230		0.558		2.84	762		0.916		4.66
300 000	0.2360		7.56		0.551	276		0.616		2.61	915		1.068		4.53
350 000	0.2750		8.16		0.595	322		0.677		2.46	1068		1.223		4.45
400 000	0.3141		8.76		0.639	368		0.7375		2.35	1220		1.376		4.38
450 000	0.3540		9.24		0.674	414		0.791		2.23	1373		1.53		4.33
500 000	0.3930		9.72		0.709	460		0.846		2.15	1525		1.68		4.28
600 000	0.4720		10.68		0.779	552		0.954		2.02	1830		1.988		4.21
700 000	0.5500		11.52		0.840	644		1.058		1.93	2135		2.295		4.17
750 000	0.5890		12.00		0.875	690		1.115		1.89	2288		2.45		4.16
800 000	0.6290		12.36		0.901	736		1.163		1.85	2440		2.60		4.13
900 000	0.7075		13.08		0.954	828		1.263		1.79	2745		2.90		4.10
1 000 000	0.7870		13.80		1.005	920		1.362		1.73	3050		3.21		4.08

Note: Actual velocity 65 miles per hr.

Table LXVII. *Stresses in wires and cables.*

B. & S.	All materials				Aluminium				Copper			
	Project. sq. in.		Hor. force lb. per ft.		Vert. force lb. per ft.		Result. force lb. per sq. in. per ft.		Vert. force lb. per ft.		Result. force lb. per sq. in. per ft.	
	Wire	Cable	Wire	Cable	Wire	Cable	Wire	Cable	Wire	Cable	Wire	Cable
8	13.54		0.989		0.400		82.2		0.445		83.6	
6	13.95		1.020		0.439		53.8		0.494		55.0	
4	14.45	14.76	1.055	1.076	0.478	0.501	35.3	36.2	0.567	0.590	36.6	37.4
2	15.09	15.60	1.100	1.140	0.536	0.564	25.5	24.4	0.678	0.706	24.8	25.8
1	15.47	15.96	1.130	1.166	0.572	0.604	19.3	20.0	0.570	0.782	20.6	21.4
0	15.90	16.44	1.160	1.200	0.612	0.656	15.8	16.5	0.837	0.881	17.3	18.0
00	16.38	17.04	1.196	1.244	0.665	0.713	13.1	13.7	0.949	0.997	14.6	15.25
000	16.92	17.64	1.233	1.286	0.725	0.777	10.8	11.4	1.084	1.136	12.4	13.0
0000	17.52	18.48	1.280	1.348	0.790	0.861	9.1	9.6	1.240	1.311	10.7	11.3
kilo cir-mils												
250		18.96		1.385		0.944		8.5		1.476		10.3
300		19.56		1.427		1.010		7.4		1.649		9.24
350		20.16		1.470		1.092		6.7		1.838		8.54
400		20.76		1.515		1.182		6.3		2.034		8.05
450		21.24		1.550		1.256		5.6		2.215		7.62
500		21.72		1.586		1.342		5.3		2.407		7.34
600		22.68		1.655		1.486		4.7		2.764		6.82
700		23.52		1.718		1.630		4.3		3.121		6.50
750		24.00		1.752		1.709		4.2		3.307		6.34
800		24.36		1.778		1.782		4.0		3.486		6.22
900		25.08		1.830		1.914		3.7		3.821		5.99
1000		25.80		1.884		2.058		3.5		4.188		5.84

Note:—Wind velocity 65 miles per hr., actual ice coating 0.5 in.
Weight of ice 57.5 lb. per cu. ft.

Finally the maximum deflection is calculated in function of the length of wire (hot), the span, weight, and area of cable by means of the equation:

$$D^3 + \frac{3\,S}{8}(S - L_H)\,D - \frac{3\,S^3 L_H W}{64\,E A} = 0\,.$$

If the supports are at different levels (fig. 288) and if b is the distance from the bottom of the span from the lowest support, the following equation is determined:

$$b = \frac{4\,D - h}{8\,D}\,L\,.$$

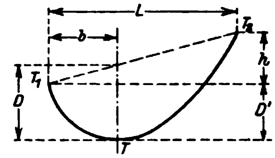


Fig. 288.

The value of D' is:

$$D' = \frac{b^2 h}{L^2 - 2\,L b}\,.$$

The tension at the lowest point is

$$T = W L \frac{L - 2b}{2h}\,.$$

The tension at the lowest support:

$$T_1 = T + W D\,.$$

The tension at the highest support:

$$T_2 = T + W\,(D' + h).$$

Tables LXVI and LXVII will be found useful in the calculation of the stresses in wires and cables, and it must not be lost sight of that a joint in a cable weakens its effective resistance by 15% more or less.

At the time of construction of transmission lines, the tension of the wires is usually measured by means of a dynamometer, or by a table calculated beforehand, determining the sag, and usually allowing a factor of safety of 4 or 5 against rupture at the lowest temperature. The sag of wires in place may be measured easily without apparatus by the following method, which consists in producing oscillations in the line by hand, at a short distance from the support.

If $g =$ constant of gravitation,
$d =$ distance from center of gravity of system to axis of oscillation,
$M_I =$ moment of inertia of pendulum,
$m =$ mass of wire, and
$w =$ number of simple oscillations per second,

the following expression is obtained

$$w = \frac{1}{\pi}\sqrt{\frac{m d g}{M_I}}.$$

Supposing the line to be of parabolic form, the moment of inertia is:

$$M_I = \frac{8}{15} m f^2$$

in which

$$f = \frac{3}{2} d = \text{distance from axis of rotation to the top of parabola,}$$
$$= \text{sag.}$$

The number of oscillations per minute will be

$$w_1 = 60\, w = \sqrt{\frac{m g \frac{2}{3} f}{\frac{8}{15} m f^2}} = \frac{60}{2\pi}\sqrt{\frac{5 g}{f}}$$

therefore the sag in feet will be

$$f = \frac{\overline{60}^2 \times 5 \times 32.16}{4\pi^2 w_1^2} = \frac{55\,890}{w_1^2}.$$

161. Influence of Wind, Snow and Temperature.

As has been said in the previous paragraph, the maximum stress which can be permitted in the wire must be assumed beforehand. It is an observed fact that the wind pressure causes an enormous stress in the wire. Generally, 40 lbs. per sq. ft. is the assumed wind pressure at a velocity of 100 miles an hour, and as is the case with a cylindrical conductor, a given wind velocity only causes half the pressure[1]); the assuming of a wind

[1]) It has been found experimentally that a wind pressure in a direction normal to the axis of a circular conductor, exerts a force on it which is equal to 0.6 of that corresponding to the projected surface, see also page 89.

pressure on a conductor, of 20 lbs. per sq. ft. is therefore amply justified. The wind pressure in function of its velocity is expressed by the following formula:

$$p = 0.0025\, v^2$$

in which

p = pressure per sq. ft. of projected cable area, and
v = actual wind velocity in miles per hour.

The wind velocities reported by the U. S. Weather Bureau stations are indicated velocities, which correspond to actual velocities as indicated in table LXVIII.

Table LXVIII. *Corrected wind velocities.*

Indicated velocity	Actual velocity	Indicated velocity	Actual velocity
10	9.6	60	48
20	17.8	70	55.2
30	25.7	80	62.2
40	33.3	90	69.2
50	40.8	100	76.2

It will be remarked that the exposed surface varies directly as the diameter, while the strength increases as the square of the diameter; therefore small conductors are more apt to suffer from wind and snow than larger ones. From the mechanical point of view, it is better to use larger conductors for power transmission. Besides carrying its own weight, a conductor must also be able to carry the weight of snow or ice which may accumulate around it in winter storms. The practice is to assume that $\frac{1}{2}$ to 1 inch of ice may cover the surface of the wire. Aluminium wire gathers less sleet than copper; this is due probably to its greasy surface.

It is obvious that in the calculations of the effect of the wind, the additional weight occasioned by the ice must be considered simultaneously, as in such a case, the wind pressure is exerted against the increased diameter of the wire due to ice. The wind pressure causes the conductor to swing to one side, and the elongation which takes place is due to the combined strain of wind and weight; in other words, the strain due to the weight of the conductor and ice is parallel to the line, whereas the strain due to wind is at right angles to the line. The strains are combined in order to determine the resultant stress in the wire itself.

If w = tension due to weight of conductor and ice, and
p = tension due to wind pressure;

the resultant tension will have for value:

$$\sqrt{w^2 + p^2}\,.$$

It may also be noted that in case of transmission of a large amount of power, the heat developed in the conductor keeps the temperature of the latter above that of the atmosphere, preventing sometimes the formation of ice.

The variation of temperature is taken at about 150° F. in the northern countries, while for the southern countries, a variation of 100° F. may be

taken as a basis for the calculations. As ice on the conductor will break off more or less with high winds, it is to be concluded that, as far as transmission lines are concerned, the strongest wind effect does not coincide with the greatest cold.

162. Aluminium Conductors.

Aluminium is being used today as conductor in overhead transmission lines, and has successfully stood the test of practical work; this metal offers many advantages compared with copper, as will be seen in the following data furnished by H. M. Hobart.

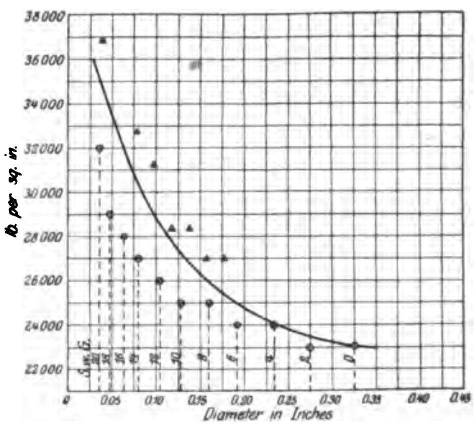


Fig. 289. Tensile Strength of Aluminium. Wires of Various Sizes.

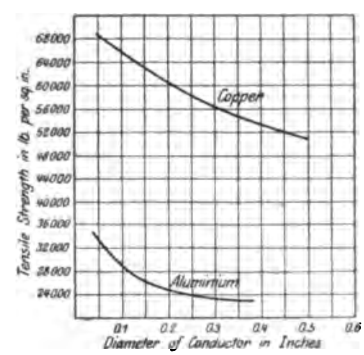


Fig. 290. Average Relation between Diameter and Tensile Strength for Copper and Aluminium Conductors.

From data supplied by manufacturers of aluminium wire, the tensile strength of hard drawn aluminium wire is shown per curve in fig. 289.

Fig. 290 shows a set of curves one of which relates to aluminium wires and the other to copper wires.

In comparing aluminium and copper, the following characteristics of both metals are to be noted[1]:

	Aluminium	Copper
Specific gravity	2.68	8.93
Relative specific gravity	1.00	3.33
Conductivity	61 to 63	96 to 99

The most complete study of the relative merits of aluminium and copper transmission lines has been made by Mr. Hobart.

Two cases were considered. In the first case, the towers carry six stranded aluminium cables, and each cable has a cross section of 200 sq. m.m. In the second case the towers carry six stranded copper cables, each cable has a cross section of 120 sq. m. m.

For the towers, M. Hobart calculates:

	Aluminium line	Copper line
Total length	21.5 m	20.1 m
Total height above ground	19.5 m	18.1 m
Load to be sustained	0.475 tons	0.940 tons
Unbalanced pull to be withstood	3.60 tons	4.80 tons

[1] The Pittsburgh Reduction Co.

The total costs of the transmission lines were calculated to be:

for aluminium	$ 5500 per mile
for copper	$ 6665 ,, ,,

In this particular case, the copper line costs 21% more than the aluminium line.

The smaller the conductors, the less is the percentage disadvantage of copper. M. Hobart observes that certain advantages inherent to aluminium have not been credited to it in the calculation. Amongst these may be mentioned the lesser freight charges due to the lesser weight, and the lower labor charges due to the greater facility with which aluminium conductors may be erected. These would of course, not greatly affect the total cost of the line, and since it would have been difficult to assess quantitative figures to them it has been deemed desirable merely to mention the omission to take these differences into account.

The cost of the copper line for equal temperature rise was found to be 28 per cent greater than the total cost for the aluminium line, as against the result of only 21% percent greater cost, at which Mr. Hobart arrived when the comparison was made on the basis that the line loss should be equal in the two cases.

Chapter IX.

Investigation of Water Power Projects. Economics.

163. Site of Power House.

All streams can generally be utilized with possible commercial results by a series of power plants, distributed from the higher plateau to the lower plains until no head is available. Suppose a river, the profile of which is shown in fig. 291. It is observed that the hydraulic development of the lower plant will cost more than the equivalent hydraulic development in the upper part of the valley; it results therefrom that the cost per HP will be correspondingly higher in the first case than in the other.

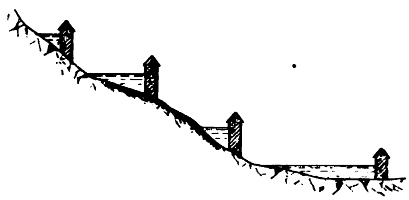


Fig. 291.

On the other hand, the çost of transportation of materials and machinery may be less when the site is located in the lower valley; in such case it would be obvious that a locality should be chosen, where transportation economy will compensate increased installation cost.

The study of a proper site for a power house naturally involves other important factors; it must not be forgotten that any particular locality implies a known distance from the power market, and therefore determines a given length for the transmission line.

It is necessary to study the various types of developments that are in use, compare the prime conditions affecting the particular case under consideration with those of existing power houses of the type which is likely to be adopted; and it must be remembered that sometimes a power house can be duplicated to advantage in another locality.

In low and medium head plants, the power house may form part of the dam, or adjoin the dam on the down stream side; in some exceptional cases the power house is located within the dam, the latter being hollow. The power house intakes should be protected from floating ice and debris; it is desirable to build a spillway or a wastegate nearby in order to take care of waste materials, and keep the intakes clear.

Special attention should also be given to the possibility of flow under and around the structure.

164. Site of Dam.

The site of a dam is governed by several factors of which its relative position with respect to the solid rock foundation is probably the most

important. The proper location will therefore be determined by the study of the most economical inter-relation between dam, canal or pressure pipe and tail race. The factor immediately influencing the site is the nature of the river bed; if this presents a knob and if the dam be built on top of this, such a scheme would obviously be the most economical. On the other hand if the bed is flat, the most economical dam would be located where the river bed is narrower. A dam built up stream with respect to the power house necessitates the construction of a canal, which at places may be very costly, fig. 292. If the dam is placed downstream, it will be higher, requiring a larger cross section area and evidently more masonry, fig. 293. However these conditions do not take in consideration

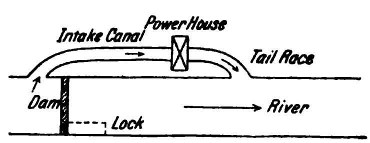


Fig. 292. Dam Up-Stream with respect to Power House.

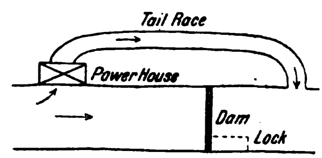


Fig. 293. Dam Down-Stream with respect to Power House.

the probable storage, in which case, the value of storage as compared with the cost of its creation would be a preponderant item. In most modern instances, however, and especially those of low head, the dam is placed adjacent to the power house or in other words, the building itself forms a part of the dam structure. Usually the power house is built at an angle in plan, a forebay is provided and a deflecting boom or submerged arches form an efficient means of preventing floating matter or anchor ice to reach the power house and block the screens. If the river is navigable, a lock must be provided, its dimensions being influenced by the amount of traffic and the size of the boats, and a guard wall should be built to prevent vessels from being drawn over the dam by the currents. The elevation of the lock masonry is governed by the high water mark of the river, due consideration being given to the depth of the water at flood times over the spillway, this being generally the type of dam used in low head developments.

165. Reservoirs.

The most important features controlling the location of a reservoir are those of a geological and topographical nature.

The selection of a site as regards water tight substrata for trench excavations for the construction of the dam, the proximity of stone which should be quarried from the neighborhood and within economical distance of the site, and material for earth embankments for the construction of either masonry or earth dams, are matters exacting attention.

In general, the ideal site for a reservoir is a region contained within a valley of relatively large area, but bounded by steep slopes, which should approach close together at one point below forming a gorge, so as to

provide a convenient dam location. It is the practice to generally study and compare two possible sites formulating for the purpose complete cost estimates for both.

Having decided on the exact location, a topographical survey of the reservoir site is then made of the area to be flooded, for which it is necessary to obtain all data as to property lines, buildings, vegetation, highways, railroad tracks, etc. From the survey map, a contour map is made, from which the computations of storage volumes for different depths can be made. The areas enclosed by the contour lines may be measured on the map by means of a planimeter, taking the average of three readings, and the volume contained between two successive contour lines is obtained by multiplying the average of the areas contained between the contours by the distance separating them. The prismoidal formula may be used where the slopes are rather flat, as the planimeter method may lead to appreciable error. The formula to be used is

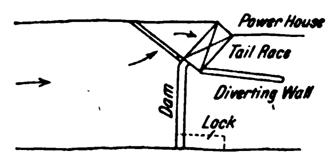


Fig. 294. Dam contiguous to Power House.

$$V = (a + 4b + c)\frac{d}{3}$$

in which

a, b, and c are the two end and the intermediate areas; and d is the contour interval.

In the case of streams carrying silt, it is necessary to make an allowance in the computation for the silting up of the reservoir, although this importance, in some cases, is problematical, as illustrated by the report made on the observations regarding the sedimentation of the reservoir formed by the Sweetwater dam.[1]

"Careful resurveys of the reservoir made by Mr. H. N. Savage, Chief Engineer, demonstrate, since it became empty, that the total filling has been about 900 acre feet, since the construction of the dam, or at the rate of 75 acre feet per annum. The total volume of water that entered the reservoir in 12 years has been 180,066 acre feet. The measured solids deposited from this water have therefore averaged a trifle more than one half of one per cent. The deposit has been almost directly as the depth, being greatest at the dam, where the depth of silt of almost impalpable fineness is two and one half to three feet. The addition made to the reservoir capacity after the flood of 1895 was 4.6 times the accumulated sediment of 12 years, or, in other words, sufficient to offset the filling of half a century."

The above statement is not general, of course, and may well be offset by the case of the reservoir formed by the dam across the Colorado River at Austin. This dam, completed in 1893 formed a reservoir of a capacity of 2,275 million cubic feet which was reduced to a capacity of 1,150 million cubic feet in 1900 on account of the extremely silt-bearing character of the stream.

[1] Transactions A. S. C. E. Vol. XIX, Page 214.

166. Prismatic Reservoir. Time of Outflow.

No inflow. In emptying a storage reservoir, a discharge under varying head occurs, and the problem may arise requiring the determination of the time necessary for discharging a certain amount of water.

Let: S = reservoir area, in sq. feet,
A = area in sq. feet of cross-section of discharge conduit,
H = head in feet at point, C, fig. 295, at the time t,
H_0 and H_1 = initial and ultimate head, in feet,
v = velocity of discharge at time t, in feet per second,
T = time in seconds required for the reservoir surface to change to an assigned level.

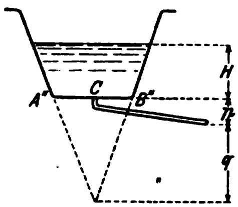


Fig. 295.

For a time dt, the discharge is $Avdt$, and the surface of the reservoir is lowered by an amount $-dH$.

Equating gives $$Avdt = -SdH.$$

But having

$$v = m\sqrt{2gH}$$

$$mA\sqrt{2gH}\,dt = -SdH$$

$$dt = -\frac{SdH}{mA\sqrt{2gH}} \quad \ldots \ldots \ldots \ldots (1)$$

$$T = \frac{S}{mA}\int_{H_1}^{H_0}\frac{dH}{\sqrt{2gH}}.$$

Integrating gives:

$$T = \frac{2S}{mA\sqrt{2g}}(\sqrt{H_0} - \sqrt{H_1}).$$

Calling

l = length of the conduit in feet, and
D = diameter in feet,

the value of m is given in Table LXIX.

Table LXIX.

$\frac{l}{D}$	m	$\frac{l}{D}$	m
5	0.77	30	0.55
10	0.64	40	0.52
15	0.61	50	0.49
20	0.59	100	0.40
25	0.57	200	0.30

Uniform inflow. In case of the reservoir receiving a uniform inflow of Q cubic feet per second, the above formula becomes

$$T = \frac{2S}{mA\sqrt{2g}}(\sqrt{H_0} - \sqrt{H_1}) + \frac{QS}{m^2A^2g} L \frac{mA\sqrt{2gH_0} - Q}{mA\sqrt{2gH_1} - Q}.$$

167. Non-Prismatic Reservoir.

No inflow. In this case, S is variable and may be represented as a function of the head

$$S = f(H).$$

Substituting in (1), paragraph 166:

$$dt = -\frac{f(H)\,dH}{m A \sqrt{2gH}}$$

Suppose

$$f(H) = aH^2 + bH + c; \qquad a, b, c = \text{const. coefficients}$$

$$T = \frac{1}{mA\sqrt{2g}} \int_{H_0}^{H_1} \frac{aH^2 + bH + c}{\sqrt{H}}\, dH.$$

Integrating

$$T = \frac{2}{mA\sqrt{2g}} \left[\frac{aH^{\frac{5}{2}}}{5} + \frac{bH^{\frac{3}{2}}}{3} + cH^{\frac{1}{2}} \right]_{H_0}^{H_1}.$$

The expression $S = f(H)$ may be found, for example, in the case of a reservoir having a pyramidal form, fig. 294, in the following manner. Calling the area

$$A''B'' = S_0$$

we find

$$\frac{S}{(H+q)^2} = \frac{S_0}{(q+p)^2}$$

or

$$S = \frac{S_0}{(q+p)^2}(H+q)^2.$$

168. Surge Tanks.

The water hammer effect produced by the sudden closing of a valve in a pipe line may be attenuated to some extent by means of a small reservoir, called also a stand or surge pipe, fig. 296. At the same time this surge tank can furnish the water necessary for a sudden demand, viz., the starting of the plant, and can also receive the water that may not be wanted at the plant, viz., when reducing load, or temporary shutting down.

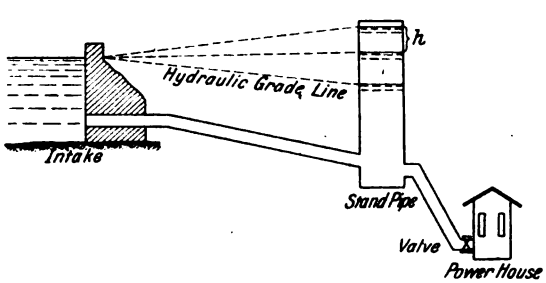


Fig. 296.

If M = mass of water flowing in the pipe of length L, and
V = velocity of this water in feet per second,
the energy of the flowing water will be

$$\frac{MV^2}{2}.$$

If the gate is suddenly closed, the water in the surge pipe will be raised to such a level as to counterbalance the energy of the flowing water.

Let h be the height of the water in the surge tank when $V=0$. The equation of equilibrium will be

$$\frac{Mv^2}{2}=\frac{Wh}{2},$$

where

$$W = \text{weight of water raised through } \frac{h}{2}$$

and which becomes

$$\frac{62.5 \cdot A_1 \cdot L \cdot V^2}{2g} = 62.5 \cdot A_2 \cdot h \cdot \frac{h}{2}$$

if the area of the pipe is called A_1 and that of the surge tank A_2. The value of h is then found to be

$$h = V\sqrt{\frac{L \cdot A_1}{g \cdot A_2}}$$

in which

$$g = \text{constant of gravitation.}$$

If the ratio $\frac{A_1}{A_2}$ is expressed by m, the above formula becomes

$$h = V\sqrt{\frac{mL}{g}}.$$

It will be of interest to note, in respect to surge tanks, and supposing the average thickness of the plate to be equal to the thickness of the bottom, that the most economical tank of definite volume is that the height of which is equal to the diameter.

Call

$$H = \text{Height of tank, and}$$
$$r = \text{its radius.}$$

Its plate area is

$$A = \pi r^2 + 2\pi r H$$

Differentiating and equating to zero

$$2\pi r + 2\pi H = 0$$

solving for H: $$H = 2r = \text{Diameter.}$$

169. Distribution of the Water between several Plants.

The distribution of the waters equally or in definite proportions between several plants is effected in different ways. In case of high head plants requiring relatively small quantities of water, the system developed by M. de La Brosse, may be used with some modifications. The water, fig. 297, is brought into a chamber by means of a siphon. If intended to serve three plants, the chamber, built of masonry, is triangular in plan, the sides being of equal length. The sides consist of spillways, and as the level of the water in the chamber will always be the same in respect to the three spillways, the discharge over the latter will be effected equally. If the water is not to be distributed equally the length of the spillway may be gauged for each specific case.

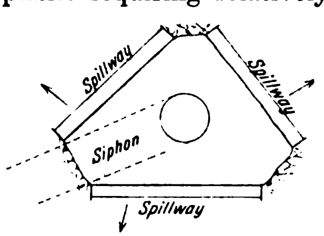


Fig. 297.

The water, passing over these spillways, may be collected in a pressure tank or collecting basin, from which pressure pipes or canals will carry it to the respective power plants.

If the distribution is to be effected by means of gates, the size of these must be determined in order to give passage to the amount of water to which the owner of the concession has right.

Calling

Q = mean discharge of river,
Q' = flood discharge of river,
q and q' = the corresponding rates of discharge of one of the plants,
H_1 = height of gate,
H_2 = head of water at top of gate opening, and
L = width of gate,

M. R. Busquet gives the following formulae:

$$q = mLH_1\sqrt{2g\left(H_2 + \frac{H_1}{2}\right)},$$

$$q' = m'L(H_2 + H_1)\sqrt{2gH_2H_1}.$$

The values of m and m', are generally 0.42 and 0.62 respectively, so that

$$\frac{m}{m'} = \frac{0.42}{0.62} = 0.68.$$

Knowing H_1, the width L may be determined, or inversely, knowing L, the height H_1 may be determined.

The quantity $(H_2 + H_1)$ determines the level of the gate sill, which is better taken equal for the different intakes. In order to attain maximum power, this height should be

$$(H_2 + H_1) = \frac{h + H_1}{3}.$$

The distribution of the water may also be effected by a common spillway, the problem being reduced to a computation of the minimum depth of overflow, this being equal to the aggregate discharge to which the other plants have right.

170. Fall Increaser.

A novel fall increaser, schematically represented in fig. 298 has been developed by Messrs. Biel and Bursh. It consists in a depression in the form of a V in the river bed or canal bottom, into which the draft tube discharges. In raising the gate, the head of the water in the forebay produces such a velocity under the gate, that in passing the draft tube opening, a suction is produced, and the momentum is sufficient to counterbalance the inertia of the tail water which remains at its natural level as shown in the illustration. If the effect of the fall increaser were neglected, the available head would be h_1; whereas by the use of this arrangement, an additional head equal to h_2 is obtained. The device is very simple and may be installed in many low head plants.

Fig. 299 illustrates Herschel's fall increaser which has been adopted in several plants.

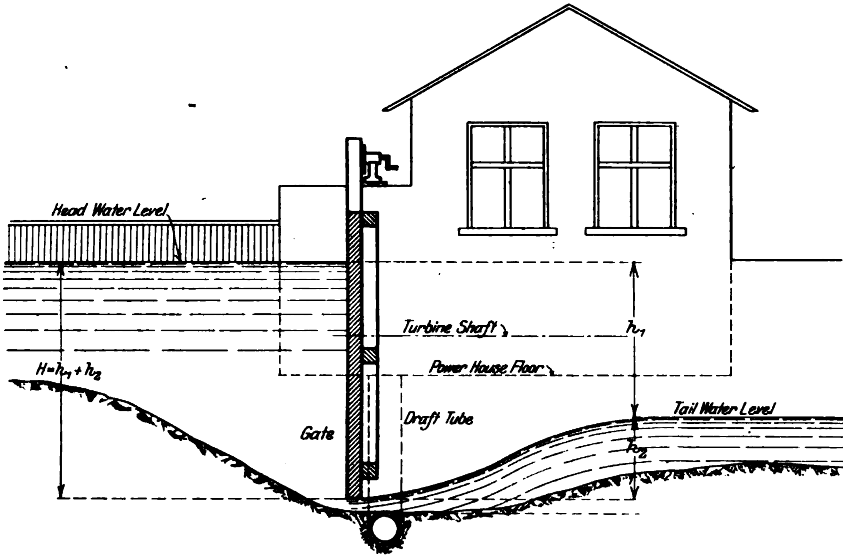


Fig. 298. Biel and Bursh's Fall Increaser.

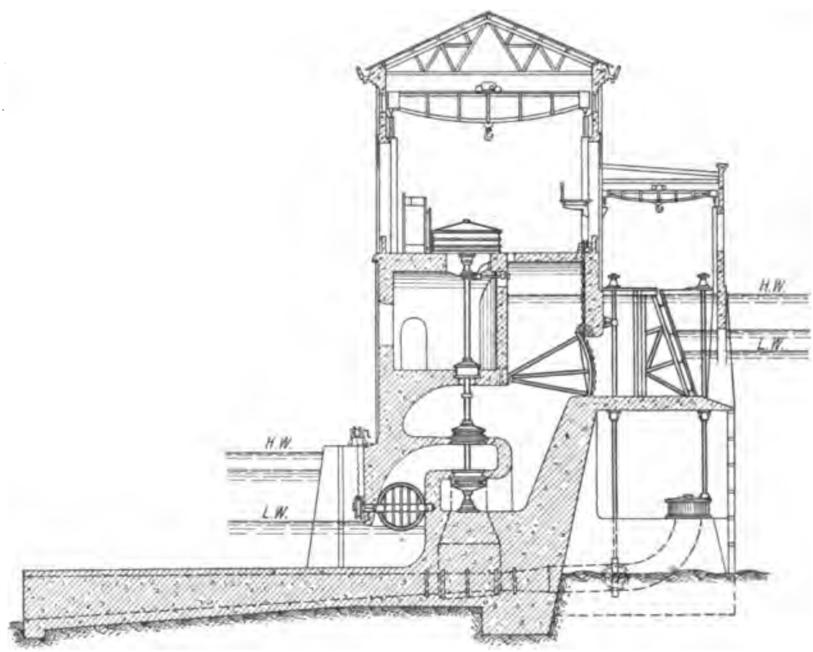


Fig. 299. Application of a Fall-Increaser to a Power Plant.

171. Study of Probable Load.

In the development of water power and the establishment of transmission lines, it is of prime importance to estimate beforehand the probable load.

Having decided on the probable route of the transmission line, it is necessary to investigate and make a list of the actual consumers of power, determining the kind and amount of power and the expenses incurred by

the consumers using it. The data taken for this purpose should indicate the address, specialty of manufacture, boilers (type and capacity), machinery (type and capacity), total load in HP and actual load in HP. The kind of coal and its cost, number of engineers and help and number of working hours per day are other factors to be noted with as much accuracy as possible.

Having compiled this data, an examination is made as to what industries could adopt electric power with profit; proposals including estimates are presented to the manufacturers, together with comparison completing the information presented in the proposal for demonstrating relative advantages and reliability of service. It will be remembered that the factor of reliability is not only an engineering problem but also a commercial one, perhaps of far greater importance to the large manufacturer than the factor of low cost.

When making such comparisons, the points to bear in mind, and which will serve to call the attention of the prospective client, are of statistical nature and refer to the motor or motors in actual use, charges for transmission, shafts and belts, fuel expense, salary and wages of help, cost of water, oil, cotton, repairs and depreciation of machinery, and rent paid for individual power plants of the factories. The advantages are then shown in favor of electrical power, ease of installation, economy, continuity of service, good voltage regulation, flexibility afforded by new system, elimination of heat and vibrations, and creation of perfect hygienic conditions for the employees.

The large industries having been duly covered, then smaller ones should receive careful attention, because although each unit of small scale manufacture may appear of little importance, nevertheless in the aggregate they may result in a large percentage of the total load secured.

172. Cost of Development.

The cost of development varies in different localities and also for low or high head plants. The location of the power plant influences the cost of machinery and materials, these naturally costing more where the locations are remote from centers of manufacture.

Table LXX. *Development costs of some American plants:*[1]

Name or location of plant	Head in feet	Horse power capacity at turbine shaft	Cost	Cost per HP	See notes below
1. Chicago Drainage Canal, Lockport, Ill.	28	15,500	$3,500,000	$225.80	d
2. Columbus, Ga.	40	9,000	450,000	50.00	c and e
3. Catawba, S. C.	25	10,000	1,100,000	110.00	d and f
4. Tariffville, Conn.	31	2,300	....	125.00	d
5. Delta, Penna	42	550	30,000	54.00	d and g
6. Lachine, Montreal	16	6,600	957,200	145.80	d and h
7. Winnipeg, Manitoba	40	25,600	4,000,000	156.25	d and i
8. Manchester, N. H.	30	6,000	....	66.00	a and j
9. Lowell, Mass.	13	....	....	110.00	a and j
10. Lowell, Mass.	18	....	....	57.00	a and j
11. Big Cottonwood, Utah	370	3,000	325,000	108.25	d and k
12. Lawrence, Mass.	....	1,000	....	67.50	a and j
13. Spier Falls, N. Y.	90	50,000	2,100,000	42.00	c

[1] From Water Power Engineering by D. W. Mead.

Table LXXI. *Development costs of some foreign plants:*

Name or location of plant	Head in feet	Horse power capacity at turbine shaft	Cost	Cost per HP	See Notes below
Zürich, Switzerland	Very low	25,300	$4,650,000	$183.90	d and l
Rhinefelden, Germany	10 to 16	15,000	1,225,000	81.70	c
Paderno, Italy	90	13,000	....	120.00	b
Champ, France	104	6,750	1,000,000	148.00	d
Dep't. de l'Isére, France	330	4,000	136,000	34.00	b
Dep't. de Jura, France	6.5	300	45,000	150.00	d
Upper Savoy, France	450	11,000	182,000	165.50	c and m
Chèdde, France	455	10,000	....	{ 30.00 42.50	a c and n
Chèvres, Switzerland	14 to 27	9,600	1,044,000	109.00	b
Kubel, Switzerland	296	5,000	1,074,000	215.00	d and o
Schaffhausen, Germany	{ 13.8to15.8 11.5to14.8 }	2,700	365,000	135.00	d and p
Gersthofen, Germany	32.8to34.4	6,000	812,500	135.00	b
Augsburg, Germany	......	9,100	1,875,000	206.00	d
Heimbach, Germany	230 to 360	16,500	2,125,000	130.00	d and q
Lyon, France	33 to 40	22,750	6,500,000	287.50	d and r
Mühlhausen, Germany	24 to 30	23,000	3,075,000	138.50	b

Notes in tables LXX and LXXI.

a The cost of water power development, not including dam.
b The cost of water power development, including dam.
c The cost of complete water power development, including electric station equipment.
d The cost of complete water power development, including electric station equipment and transmissions lines.
e Mostly 12-hour HP distributed to adjacent mills at the generated voltage.
f Severe climatic and river conditions during construction.
g Very favorable location; cheap timber dam; transmission line only 5 miles long.
h Includes extra real estate investment.
i Expensive canals in rock, and very extensive concrete construction.
j Factory installation.
k Pelton wheels and 1500 ft. wood-stave pipe line.
l Four interconnected plants; including also steam auxiliary.
m Not including 5000 HP necessary steam auxiliary.
n Not including dam.
o With 1000 HP steam auxiliary.
p Two interconnected plants.
q 15 mile transmission line.
r 12 mile feeder canal.

The estimated cost of a 200 mile power transmission project in Sweden and Denmark including submarine cables line, 3.4 miles in length is tabulated as follows: (pre-war estimate)

Estimated Cost of Generating and Transmitting Electric Power, Trollhättan-Copenhagen Project.

	Alternating Current		Direct Current	
	Cost of Installation	Annual Expenses	Cost of Installation	Annual Expenses
Generating station.				
Two 10,000-hp. turbines and auxiliaries	$ 81,000	$ 7,600		
Three-phase generators	70,500	6,600		
Transformers at Trollhättan	31,500	2,950		
Switch gear	27,000	2,650		
Wages, oil and supplies	—	3,200		
	$ 210,000	$ 23,000		

	Alternating Current Cost of Installation	Alternating Current Annual Expenses	Direct Current Cost of Installation	Direct Current Annual Expenses
Generating station (continued):				
Changes for turbines			$ 40,500	$ 3,400
Isolating machinery			2,700	200
			$ 43,200	$ 3,600
Four 5000-hp. turbines			$ 81,000	$ 7,600
Direct-current generators and auxiliaries			216,000	21,400
Wages, oil and supplies			—	3,200
			$ 297,000	$ 32,200
Transmission line:				
Iron poles, including erection, at $ 1350 per mile	$ 275,000	$ 27,200		
Insulators, at $ 390 per mile	80,000	7,900		
Iron cable, at $ 87 per mile	17,900	2,500		
Copper conductors, at $ 1525 per mile	310,100	21,000		
Wages for linemen	—	2,200		
	$ 684,000	$ 60,800		
Poles, including erection			$ 125,100	$ 13,000
Insulators			31,900	3,200
Iron cable			17,800	2,500
Copper conductors			166,200	11,300
Wages for linemen			—	2,200
			$ 341,000	$ 32,000
Three transformer stations:				
Buildings	$ 40,500	$ 3,400		
Transformers	81,000	7,600		
Instruments	30,500	3,000		
Auxiliaries	40,500	4,200		
Wages and supplies	—	4,000		
	$ 192,500	$ 22,200		
Submarine cables:				
Four cables, each 3.4 miles, at $ 5780 per mile	$ 78,600	$ 14,200 (items 1–4)		
Loading on steamer, laying and splicing	20,000			
Freight and marine insurance	5,400			
Anchorage at cables	9,500			
	$ 113,500	$ 14,200		
Two cables, each 3.4 miles, 150,000 circ. mils			$ 24,000	$ 5,400 (both items)
Laying of cables, etc.			18,900	
			$ 43,500	$ 5,400
Frequency-converter station:				
Three frequency changers, 25 cycles to 50 cycles; total 11,600 kw	$ 122,000	$ 11,400		
Switch gear	13,500	1,300		
Wages, oil and supplies	—	3,200		
Incidentals	27,000	3,200		
	$ 162,500	$ 19,000		
Direct-current motors and auxiliaries			$ 221,000	$ 22,000
Three-phase generators			51,500	4,800
Switchboard and switch gear			10,800	1,100
Wages and supplies			—	3,200
Incidentals			54,000	6,200
			$ 337,300	$ 37,300
Incidentals	$ 231,500	$ 24,100	$ 140,000	$ 14,400
Total	$ 1,593,000	$ 163,300	$ 1,202,000	$ 125,100
Total per kw	—	$ 15.66	—	$ 11.16

173. Cost of Operation.

The annual cost of operation of hydro-electric plants is distributed as follows:

Administration and operating expenses,
Maintenance and repairs,
Depreciation,
Interest, insurance and taxes.

From the time that a plant is being put in operation, all the materials and physical property begin to deteriorate, a deterioration which is accumulative in character, and which continues until the property becomes unfit for further service. This deterioration takes place at different rates for different materials and physical properties, and each material must be replaced as soon as its worn condition requires. Expenses are therefore created which are ultimately borne by the customers, and these charges being continuous and constant, they must be considered a part of operating charges. Therefore a certain amount is placed aside each year in order to replace useless material, and these amounts are adjusted or proportioned to the life of the materials concerned. Depreciation being therefore an amount destined to keep intact the capital invested in the original development, it is a most important factor, and must receive due consideration on the financial side of all industrial enterprises.

Hydraulic development:

Maintenance.

Dam, hydraulic development and construction	2.1 %
Pressure pipes and accessories	2.5 %
Turbines and mechanical equipment	6.0 %

Depreciation.

Dam (30 years)	2.1 %
Pressure pipes and accessories (20 years)	3.72 %
Turbines and mechanical equipment (12 years)	7.0 %

Electric development:

Maintenance.

Electric machines and material	5.0 %
Transmission lines	2.0 %
Buildings	2.0 %

Depreciation.

Electric machines and material (12 years)	7.0 %
Transmission lines (20 years)	3.72 %
Buildings (30 years)	2.1 %

As the estimated revenue will not attain its value before some years of operation, it is best to assume that only $\frac{2}{5}$ of it will be realized after the first year, increasing each year for the first 4 years after which time it is fair to expect the plant will be running at full capacity and therefore producing the expected income.

The following is the estimate of the cost of operation of the Chicago Sanitary District hydro-electric plant:

Total cost of development and transmission	$ 3,500 000.00	
Estimate of operation cost.		
Interest on investment at 4 per cent	140,000.00	
Taxes on real estate, buildings etc.	7,620.00	
Depreciation on buildings at 1 per cent	3,650.00	
Depreciation on water wheels at 2 per cent	2,027.32	
Depreciation on generators at 2 per cent	1,824.60	
Depreciation on pole line at 3 per cent	2,020.50	
Depreciation on other electrical appliances at 3 per cent	3,995.52	
Total fixed charges		$ 161,137.94
Operating Expenses.		
Power and sub-station labor	63,240.00	
Repairs to machinery and buildings	3,700.00	
Incidental expenses	1,200.00	
Operating Lawrence avenue pumping station	43,960.00	
Operating 39th avenue pumping station	120,380.00	
Interest on investment 39th avenne pumping station	15,599.76	
		248,079.76
Total cost to sanitary district		$ 409,217.70
Capacity 15,500 HP. Cost per HP. per annum		$ 26.40

174. Financial Considerations.

When reporting on the feasibility of a development there are a number of essential points to bear in mind. They may be summarized as follows.

Not only the durability of the structure itself is a subject requiring careful attention, but also judgement should be used regarding its continued usefulness independent of its durability; methods and processes may be introduced which would affect its future applications disfavorably or otherwise. On one hand there may be local factors impeding the prompt application of available power, while on the other hand, the facility of a relatively cheap power may foster new applications thereof in the locality.

Especially where varying types of structure are subject of consideration, it is recommendable to use duplicate methods of cost comparison for the purpose of indicating economy, either in general or in the long run of some one of the types treated. This two-fold method comprises on one hand the consideration of annual upkeep, running expense, and depreciation as well as the original cost as a capitalized value, and on the other hand the original cost only as a capitalized value considering the respective items of interest, upkeep, running expense and depreciation as an annual charge. The problem thus treated is evidently not an easy one, as interest rates, material- and operation-costs and even methods of construction change in the course of time. If a sinking fund charge is one of the factors, it is now generally admitted that as a rule the engineer need not take this into consideration in a duplicate cost comparison, this factor being fairly covered by a calculation of depreciation in the permanent structure; there are however a few exceptional cases, where the sinking fund charge should also be taken into account viz., in improvements of supplies for large cities.

Where regularity of service during every day of the year is an indispensable requirement, due provision must have been made against all possibility of interruptions, by means of suitable auxiliary power, storage or pondage, and the plant itself furnished with extra units and dupli-

cate parts; even where the market is not such an exacting one, similar provisions are often very recommendable.

Although undeveloped water power is apparently waste, its utilization does not necessarily imply profit; the market for the power developed must be extensive enough to consume quantities which will cover all annual charges, and still yield a fair return to those who fathered the venture, and of course the selling price of that power must be such as to defy the competition of power derived from any other source. It will be remembered that the plant's annual cost is a fixed charge whether the consumption be large or small, and that many developments have been made at a large and unwarranted cost and later involved in commercial failure because the conditions were not such as to present a suitable scope to the enterprise.

The market for power must be considered at the inception solely from its minimum and immediate possibility, while in the primitive construction, especially where there is dam or other river protection work provision may have duly been made for increased power production as soon as the market demand will justify same. Nevertheless provisions may be made at the inception of an enterprise for fostering the local demand; most public service corporations to-day tend to facilitate in every possible manner the increased consumption and novel applications of power. It is often not enough that power merely be produced at a relatively cheap rate in order to promptly interest the public in its profitable applications, and any prudent policy adopted for thus creating and extending the market falls quite within the legitimate sphere of a development enterprise.

Finally it must not be forgotten that occasionally development ventures have been involved in commercial failure through serious errors in cost estimates; unforeseen contingencies must be provided for in the estimate in each item constituting a factor in the plant's financial life.

The engineer who is capable of designing an economical plant in a favorable site should also be able to give the above points due consideration, therefore having no difficulty in judiciously reporting on its feasibility.

175. Auxiliary Power.

It is of fundamental importance to maintain continuous power, which may be considerably more than the minimum flow of the stream. The question of auxiliary power equipment becomes then a matter of serious consideration, not only for tiding over the low water period, but also for the sake of continuity of service, providing reserve capacity in case of a possible breakdown. Naturally the design of an auxiliary steam station is different from the design of an ordinary water power plant. The period of auxiliary operation lasts only two or three months per year, and consequently the fixed charges bear heavily upon the cost of the output generated by steam. In the actual design, simplicity. low cost and convenient location are the factors of fundamental importance. It will be found advantageous in most cases, to locate the auxiliary plant at the receiving end of the line, instead of locating it near the water power plant. One reason for this is that fuel is obtained cheaper at the receiving end, and another reason is that the auxiliary plant is not put

out of service by the failure of the transmission line. In ordinary low water operation the steam plants turn energy into the system at or near points of very large consumption, so that the line losses are relatively small. In case of breakdown of one of the long transmission lines only a portion of the auxiliary plant service, at worst, can be put out of service, so that operation can go on throughout most of the system.

In case of resorting to pondage, in order to subsequently use the stored water to increase low water flows, it is customary to determine from a mass curve, the flow which may be maintained continuously in an average dry year.

Rippl's[1] method may be used for the purpose of such computations. From the hydrograph of the water shed for several years, the period of extreme low water is selected. For this period the observed or estimated flow of the stream for each month is reduced by the loss due to evaporation, leakage, etc.

The result will represent the net yield available for power purposes. Add together the yields from the beginning to each month in succession; then from these figures construct a curve *OA*, fig. 300 in which the abscissa of any point is the total time from the beginning of the selected period, and the ordinate is the total net flow during the time represented by the abscissa. The inclination of the curve at any point is thus equal to the rate of the net flow, a minus inclination, as at *B*, representing a negative flow. Now in like manner plot a curve of consumption, *OC*, which may ordinarily be assumed to be a straight line, as the variation month by month is a refinement hardly warranted by the accuracy of the other data.

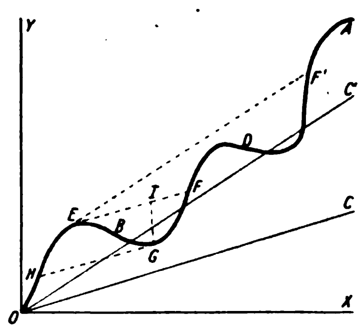


Fig. 300.

The ordinates between the lines *OA* and *OC* will now represent the total surplus from the beginning, and where the two lines converge, as at *B* and *D*, the yield is less than the demand, and conversely. The greatest deficiency occuring during any dry period *B* is found by drawing *EF* parallel to *OC* and tangent to the curve; and the amount of it is given by the maximum ordinate *IG* drawn from *EF* to the curve. The deficiency for any other dry period is found likewise, and the maximum so found is the storage volume required to supply the demand *OC*. The time during which the reservoir would be drawn down below high-water line would be represented by the horizontal distance between *E* and the point of intersection *F*. In like manner the storage capacity for any other rate, *OC'*, may be determined by measuring from the tangent *EF'*.

If the tangent from any summit does not intersect the curve, it indicates that during the period investigated the supply is not equal to the demand; and to insure a full reservoir at the point *E*, for example, it is necessary for the parallel tangent drawn backwards from *G* to intersect the curve at some point *H*. In investigating various dry periods it is therefore necessary to begin the curve a year or two back of the dry

[1] Proceedings Inst. C. E. Vol. LXXI, page 270.

years to insure the accumulation of surplus water. When actual stream measurements are to be had covering a series of years, it is best to consider the entire period.

If the yield is to be limited by the time during which the reservoir is to be drawn below high-water line, the rate of supply and corresponding storage can be determined by finding by trial the line of lowest slope which shall be tangent at a summit, and whose horizontal projection equals the time specified. If the storage is fixed and it is desirable to know what amount of water the area will yield at a constant rate per month, the rate is found by drawing the lines from various summits, which shall have their maximum ordinates equal to the given storage. The rate is given by the line of least slope.

If the case is one where the consumption cannot be assumed as uniform, the line *OC* will be a curve, and the desired information can be more easily gotten by replotting the ordinates between the demand and supply curves — the accumulated surplus — from a horizontal axis, as in fig. 301. Storage volumes, etc., are then found by drawing the tangent lines *EF*, etc., horizontally.

It must be noted however that the benefit expected of the use of a reservoir consists mainly in the insurance that it affords in keeping the flow of the stream at a determined rate.

A method devised by A. H. Perkins[1] and called by him the "utility" method, is a method which permits of the use of practically all of the stored water every year without unduly sacrificing the "insurance" feature. He says however that such a method requires fore knowledge of the amount and distribution of the run-off of the year in order that the reservoir may be just empty when the stream flow has risen to the rate for which it is proposed to regulate as a minimum. Otherwise the reservoir may become exhausted before the stream rises, and the insurance feature will be sacrificed. Practically, however, the utility method appears feasible for the reason that the economic development of such streams as he is considering as determined from the average of actual installations for such streams as the Hudson, Genesee, Oswego and others in close proximity to large centers of population, is such an installation as can be run at full capacity about 60% of the time reckoned over a long period of time. Of course, this can be only an average, conditions at each plant determining the actual limit of economic development. The fact that it is the present limit of average development established by commercial practice extending over a long period of time precludes the necessity of entering into any computations of what the theoretical limit would be. This development beyond the low-water power of the stream and the auxiliary power or curtailment of output which it implies gives a considerable margin within which the flow may be manipulated and still be within the wheel capacity of economic development and above the minimum insured flow. Thus, if it is found that abundant fall rains

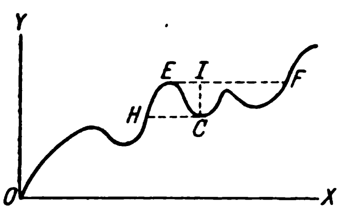


Fig. 301.

[1] Effect of Storage Reservoirs upon Water Powers. Eng. Rec. Vol. 63, No. 15.

and other conditions indicate large natural flow near the end of the period of use of stored water the auxiliary plants may be closed down completely. Or if the indications are that the reservoir will be heavily taxed to last through, auxiliary plants may be started and run at full capacity and the use of stored water correspondingly cut down. The utility method implies for its full scope of application the leeway mentioned above arising from development up to the "60% point." It thus assumes

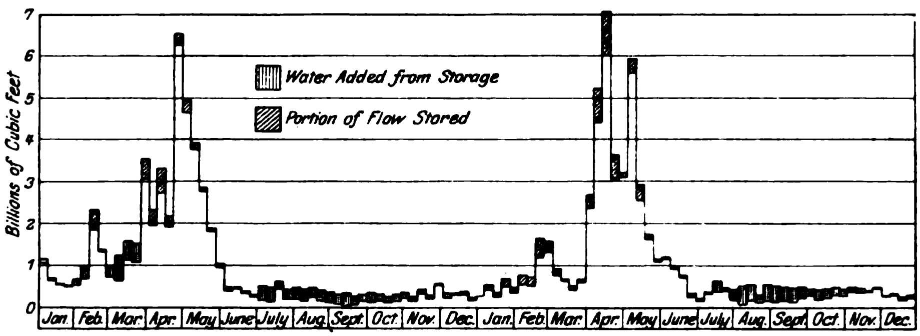


Fig. 302. Weekly Hydrography of Hudson River at North Creek.

that there is always sufficient demand for power to absorb any additions and render further development after regulation as desirable as before regulation. Although during the period of readjustment immediately following the reconstruction of a large storage reservoir for power purposes, the market may be disturbed, still ultimately this assumption will come close to the facts. It will appear from a study of the diagrams that with

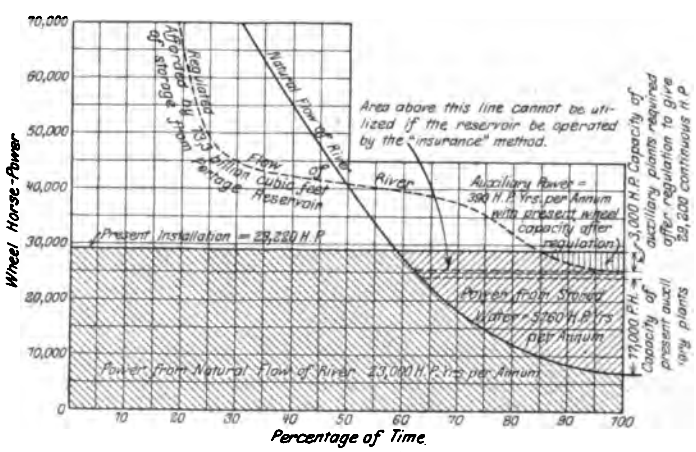


Fig. 303. Effect of Regulation; No Capacity Increase.

such regulation of a stream as is within the bounds of commercial practicability, the same money invested in auxiliary plant will produce greater returns when used in connection with the regulated flow than it will when used in connections with the unregulated flow.

Further evidence that the utility method of operation of storage reservoirs is the proper method when making studies of the expected results is furnished by the hydrograph shown in fig. 302. This is a hydro-

graph of the Hudson River at North Creek. The shaded portions show the water released from Indian Lake reservoir under actual working conditions. The Indian Lake reservoir is operated as desired by the power owners, and the hydrograph indicates that the result is at least as close to the utility method as it is to the insurance method.

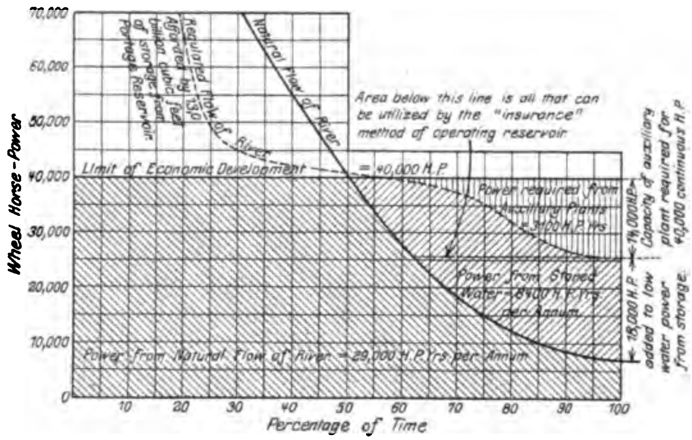


Fig. 304. Effect of Regulation; Capacity increased.

In order to study the various phases of results to be derived from storage of flood waters for power purposes, and their use in accordance with the utility method, M. Perkins devised the graphical method illustrated in figs. 303 and 304.

For this study there were available seven years of stream flow record. The curve marked "natural flow of river" was platted in an obvious manner from this record. The percentage of time that any given horse

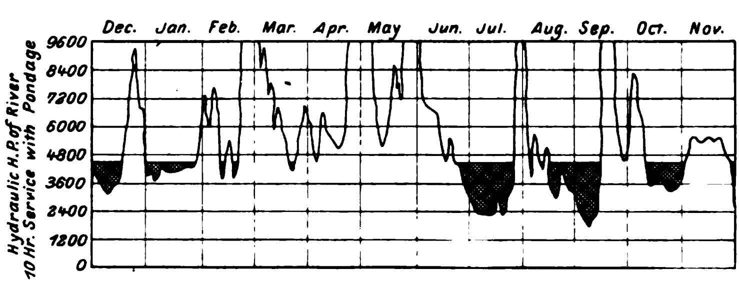


Fig. 305. Hydrograph showing Auxiliary Power necessary to maintain 4450 Ten-hour Horse Power at Sterling, Ill.

power would have been operated at full capacity during the 7 years may be read at the bottom by proceeding from the given number over horizontally to the curve and thence downward to the bottom. Thus an installation of 40,000 HP could have been run at full capacity 50% of the time from March 1904 to Feb. 1910 inclusive. This has been assumed in this case to give a long term average for the stream. On the mass curve of the stream the regulated flow obtainable from full use of the available storage was then determined for each year and from these values was platted the curve marked "Regulated Flow ol River", etc. This curve

shows the relation between power and percentage of time that exists after regulation in the same manner that the "Natural Flow of River" curve shows the relation existing before regulation. The line showing "present installation" is the sum of all the wheel capacities now working under the head being examined.

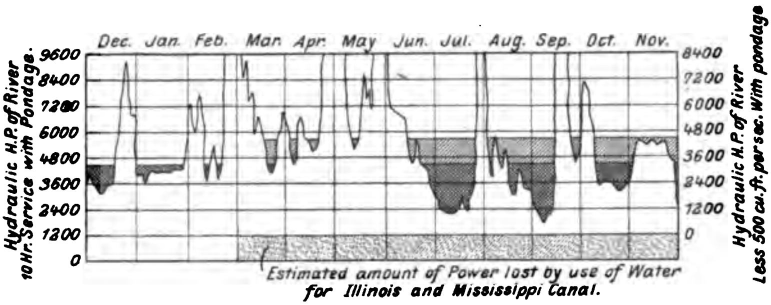


Fig. 306. Hydrograph showing Auxiliary Power needed to maintain Capacity of Wheels and Probable Increase due to Diversion of Water for Illinois and Mississippi Canal.

In order to avoid confusion two charts are shown, Fig. 303 embodying the results to be expected under present conditions, and also after regulations, but, without increase in wheel capacity of installations, and fig. 304 showing conditions after regulation and an increase in wheel capacity to 40,000 HP.

An inspection of the diagrams will show that in this particular case, operation by the "insurance" method will sacrifice about half the energy available from storage and would not raise the limit of economic development. It will also be seen that, assuming the utility method is used, a slightly smaller auxiliary plant will be required to develop the stream to the same relative economic point, and

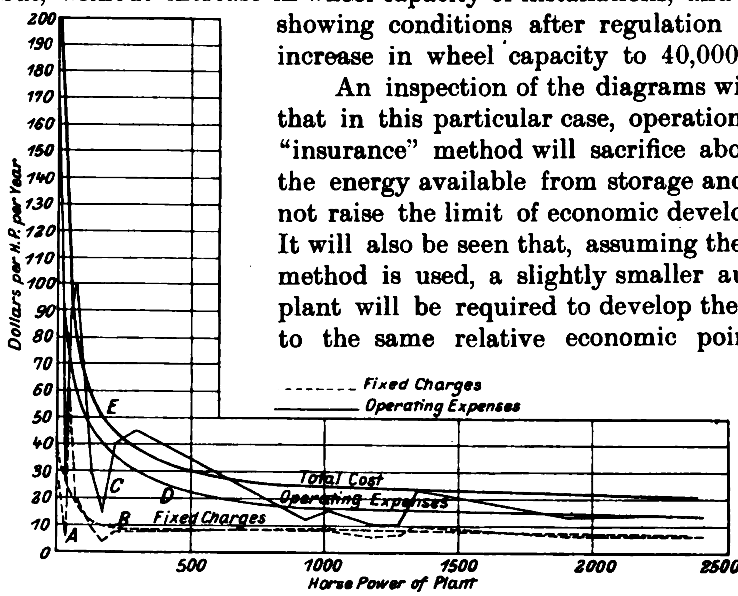


Fig. 307. Cost of Steam Power per Horse Power per Annum in Various Plants.

that the total energy that the auxiliary plant will have to develop to produce continuous conditions will be considerably less than that required from the auxiliary installations to produce continuous results under present conditions. It may be that in practice some portion of these ideal results may not be available, but it is believed that it would be possible to very closely approximate them.

The computation of the capacity and amount of auxiliary power without storage to maintain at a determined rate can be investigated

from the hydograph of the daily flow of a river as illustrated in figs. 304 and 305.

The cost of the development of steam power is a function of the cost and character of the coal used, the machinery employed, the character of the load and the number of working hours of the plant per annum. Fig. 307 is a result of the observations made by Mr. H. A. Foster of 22 different plants; table LXXII, and the curves show the power consumption per year and its cost including running expenses and fixed charges.

Table LXXII. — *Showing average power developed and its cost per HP in 22 steam power plants.*

Output		Operating expenses, per HP	Fixed charges, per HP	Total cost, HP per annum	Cost per HP hr., cts.
Average HP developed	No. of days per annum				
12.4	361	$ 147.93	$ 25.40	$ 173.33	5.648
20.9	365	123.12	28.42	151.54	1.868
21.5	361	90.47	17.80	108.27	2.918
32.9	330	22.56	5.83	28.39	0.832
36.7	365	137.25	96.70	233.95	2.811
42.4	365	86.38	63.20	149.58	1.708
53.0	309	56.94	19.51	76.45	1.596
58.8	365	97.30	33.82	131.12	1.613
70.4	365	101.69	20.78	122.45	1.641
129.3	365	30.14	9.41	39.55	0.871
166.7	313	15.19	4.47	19.66	0.639
173.0	313	22.66	5.83	28.39	3.333
210.9	290	40.33	7.86	48.19	.693
296.7	297	45.56	7.81	53.37	.749
926.0	307	11.73	8.77	20.50	.691
1010.8	306	15.70	7.74	23.44	.794
1174.8	306	10.19	5.50	15.69	.531
1278.7	293	10.49	6.23	16.72	.590
1345.5	365	23.28	9.42	32.70	.820
1352.0	365	33.03	29.41	62.44	.713
1909.7	306	13.40	6.63	20.03	.677
2422.	306	15.67	6.73	22.40	.757

176. Study of Power Development.

Without losing sight of probably desirable future enlargements, power development should be estimated conservatively, i. e.: to a point below the probable commercial maximum demand. If the demand for power is relatively large and if its cost from other sources are sufficiently heavy, pondage or other auxiliary service may be desirable. The feasibility of such a project however must be carcefully investigated by itself.

Estimates of cost should be made with the utmost severity insofar as the tendency to undervalue expense is concerned. Reasonable allowances should be made for unforeseen and possible contingencies, remembering that if such estimates result greater than the real outlay, the owners are never dissatisfied, whereas if the contrary results, serious complications may arise and the reputation of the engineer may suffer. Therefore the estimates of cost should be made with the certainty that the items may be reduced but never exceeded in construction.

The report must be made in such a manner that any good business man can clearly understand the basis on which the engineer's opinion

rests, and the degree of certainty with which the expected result can be secured. While the technical details may be beyond the range of vision of others than engineers, if the logic of the situation is duly appreciated by the informant, he can clearly, accurately and concisely expound the feasibility of the project in all its bearings. The statements, findings and recommendations of the main body of the report should be supplemented, supported and confirmed by plans, drawings, hydrographs, tables, any other voluminous data, and any elaborate or minute discussion of a technical nature.

The data, of course, should embrace every topic of investigation arranged in the most convenient form.

In regard to topography and geology, if sufficient run-off and rain-fall data is available, the study of conditions affecting run-off is not of particular importance. Otherwise, it must be made, and at the same time all other sources of information utilized which will in any way assist the engineer to comprehend the problem presented and its possibilities and probabilities.

In studying flood flow, the maximum must be determined in order to intelligently design the dam and other flow control works; usually the information is available in the respective stream valley and can be safely approximated after establishing the rating curve and observing relation of gauge heights to neighboring high water marks.

The probable limit of the backwater flow line under variable flow conditions should be ascertained in a fairly reliable manner by a carefully made topographical survey extending upstream from dam site.

Upon considering the practicable limit of flood height upstream and height of water surface at dam due to various sections and lengths of spillway, together with the foregoing data will usually establish dam height, and at same time the desirability or necessity of flood gates, adjustable crest, flashboard, or means for controlling flood height. Then power development can be calculated according to the determined variability of head and flow conditions.

The study of storage and pondage often merits the attention of the engineer as being of great importance; these conditions materially affect available amount of power, and not only the survey immediately above dam site may be consulted, but sometimes special surveys in country behind the backwater effect may be warranted.

Due consideration should be given to the study of probable load curve, the demand for power throughout the day and if variable throughout the year having a most important bearing on the design of the plant. The variability in use of the power naturally will enter into consideration of stream flow and especially the desirable condition of pondage. A census of probable distribution of power should be prepared with utmost prolixity and care although reliability in the estimates will involve considerable difficulty; most plants ignore the definite amount of power consumed, few using recording instruments of precision for this purpose. Where steam engine indicators are in use, annual coal consumption, boiler service, heating requirements and engine conditions are known, careful investigation will enable the making of a fairly close estimate of the power used. But it must not be forgotten that in non-electrically operated establishments the normal requirement is larger than where individual motors are used on each machine.

Chapter X. Description of Hydroelectric Plants.

177. Limmat River Hydroelectric Plant, Switzerland.

(Head 14.4 to 17.4 ft.)

This plant has been constructed near the town of Aue, at the side of an old water-power spinning mill.

Dam. The dam produces a head varying from 14.4 to 17.4 ft. The Limmat River is fed mostly by mountain springs, and its flow varies from 25,200 cu. ft. per second in summer to 63 cu. ft. per sec. in winter. The plant has been designed for three units of 680 kw. capacity.

The turbines, see fig. 308, are of the Francis double runner type, 70.8 in. in diameter, using 360 to 400 cu. ft. per sec. per runner at 75 r. p. m. The water from each runner discharges into a common draft tube. Each turbine drives a generator of the umbrella type, furnishing current at 2200 volts. Oil is used in the step bearing at the base of the vertical shaft, the necessary pressure being supplied by a 3-cylinder pump. The chamber beneath the lower wheel may be emptied by a pump and by simply closing the inlet and outlet openings.

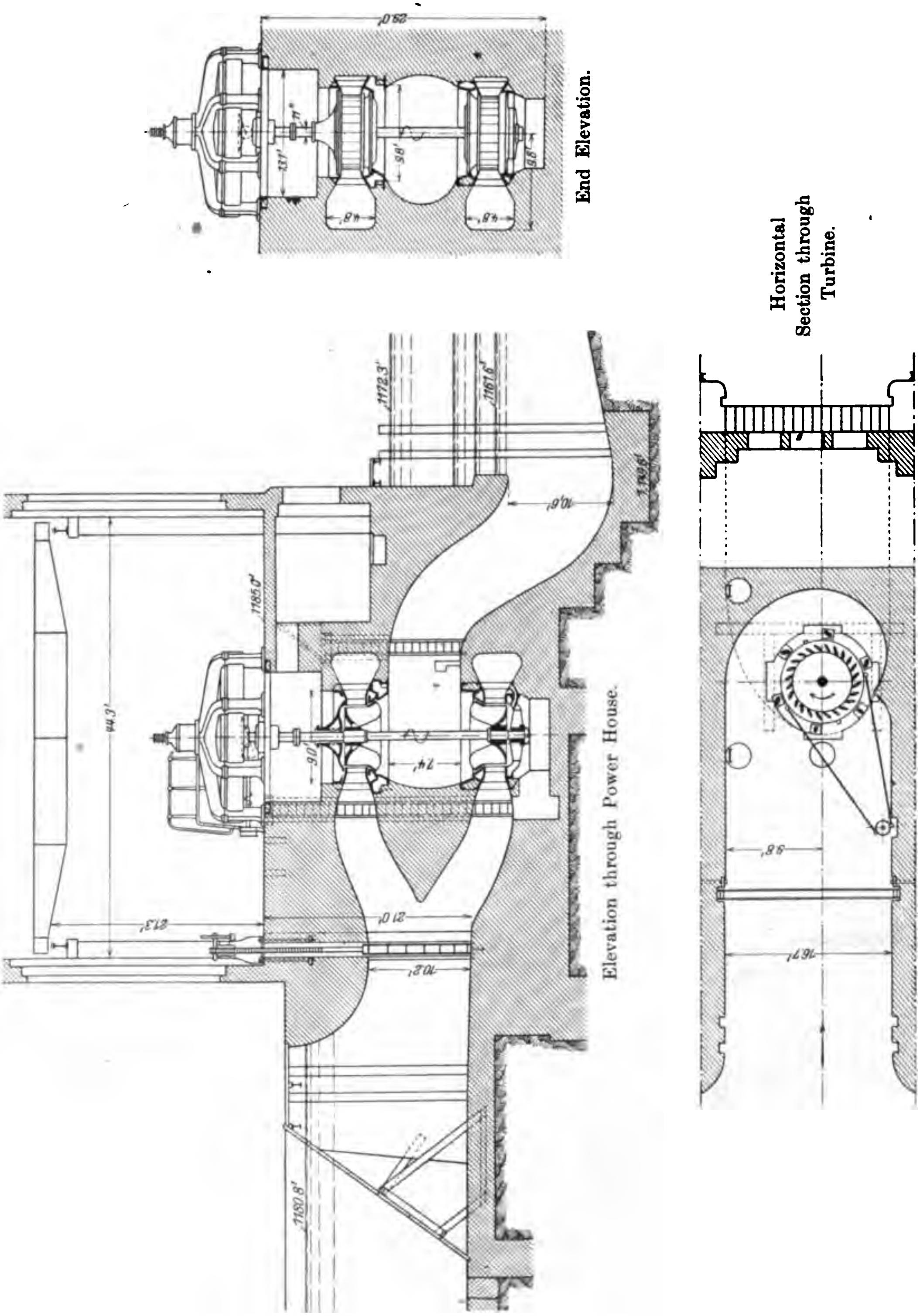
178. The Hydraulic Plant of Tuilière, France.

(Head 20 to 40 ft.)

One of the most important plants of France put in service lately is the Tuilière plant, fig. 309, located on the Dordogne River in the south western part of the country. The flow of the river is irregular, varying from 1080 cu. ft. per sec. to 135,000 cu. ft. per sec.

Dam. The type of dam adopted consists of a series of gates, and it is built at right angles across the Dordogne. In the present plant are included the dam and turbine plant, a steam turbine auxiliary and an adjoining transformer building. The total capacity of the plant is 35,000 HP.

The dam has the form of a viaduct with plate girder spans supported on masonry piers. Between the piers are mounted one-piece metal gates which slide in grooves and are worked by motor-driven hoists. The gate sill lies 3′ - 3″ below low water level, and the top of the gate when lowered lies 39′ - 7″ above low water level. A fishway, 23 ft. wide, is located between the left-hand abutment and the last pier.



End Elevation.

Elevation through Power House.

Horizontal Section through Turbine.

Fig. 308. Arrangement of Turbines in Power Plant at Aue, Switzerland.

Power house. The building is 223 ft. long and 40 ft. wide. The water is admitted to each turbine by means of a metallic gate (fig. 310, 311, 312) 22'-2" wide and 15'-2" high. These gates are operated by hydraulic lifts which work at an oil pressure of 25 atmospheres. There are

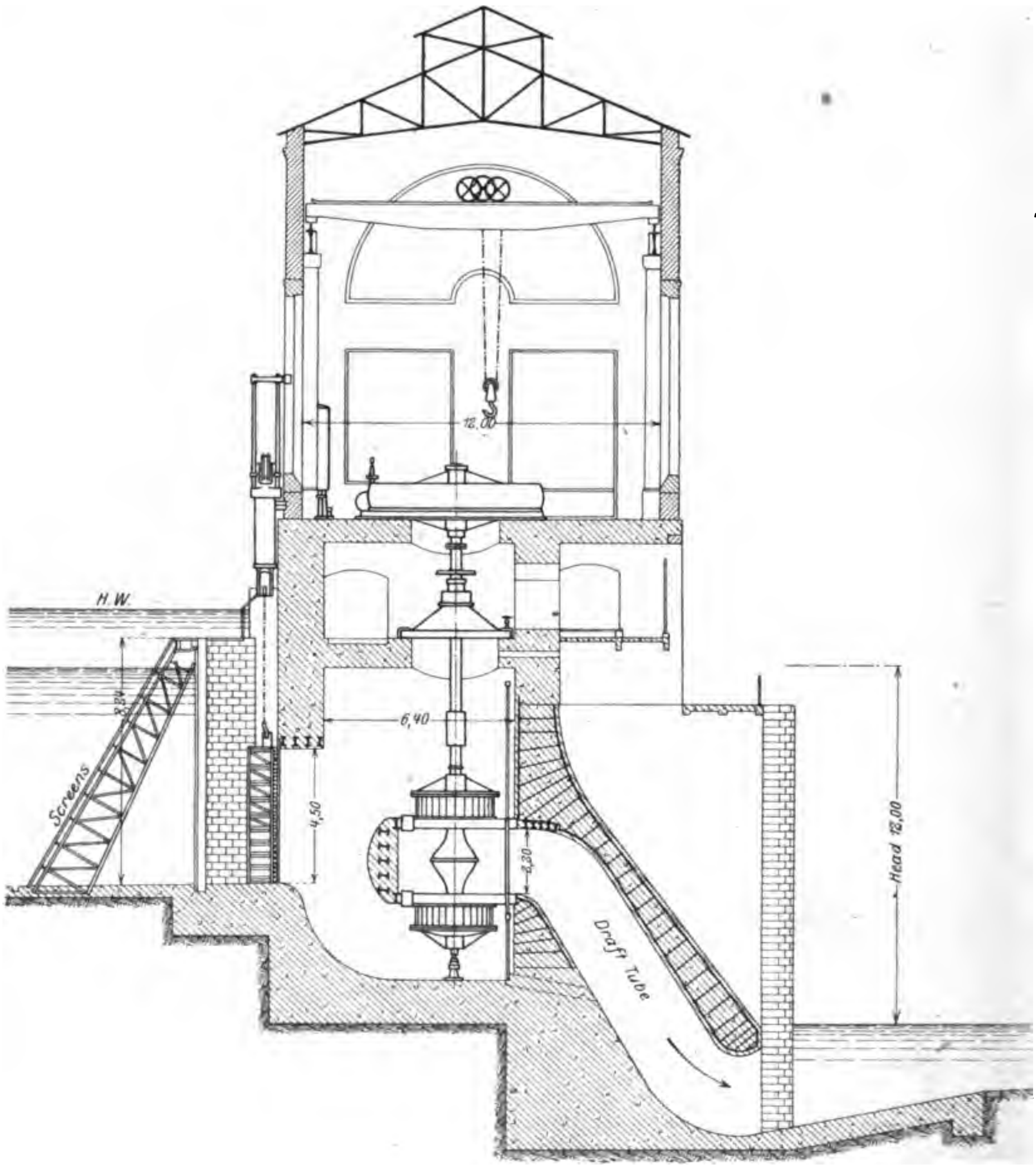


Fig. 309. Hydroelectric Plant of Tuilière (Dordogne River).

9 vertical shaft turbines of the double Francis type. They are designed to operate with a head ranging from 20 to 40 ft. Each of the turbinehalves consists essentially (fig. 313) of two fixed directing wheels with combined fixed and movable vanes. Each directing wheel has 20 vanes mounted on a cast iron crown-piece, 11'-6" in diameter. The revolving wheels have

6′-7″ outside diameter, and the common shaft on which the wheels are mounted has a diameter of 12″.

On the turbine shafts are mounted 5500 volt three phase alternators.

The auxiliary steam plant contains at present two 5000 HP Curtis turbo-alternator sets. The switchboard is located on a gallery of the building and commands all the groups of the plant.

The transformer house is a separate building located on the downstream side, and the power is transmitted through the region of Bordeaux at a tension of from 13,500 to 50,000 volts.

179. Sturgis Plant, Michigan.

(Head 25 ft.)

This plant is built on the St. Joe River, the flow of which varies from a minimum of 300 cu. ft. per sec. in dry season to 4000 cu. ft. per sec. in extreme flood.

Dam. This structure is of unusual design. It is 300 ft. long and made up of fifteen 40° inclined reinforced concrete arches supported by concrete abutments 20 ft. apart.

Power house. The power house consists of a concrete building 50 ft. × 35 ft. containing two 844 HP. Allis Chalmers turbines, each of which drives a 550 kw., 2300 volt 60 cycle 3-phase alternator mounted on the same shaft. One single bearing located at the floor level carries the entire weight of the rotating element. There is only one exciter unit of 40 kw. capacity which is driven by a separate wheel, the

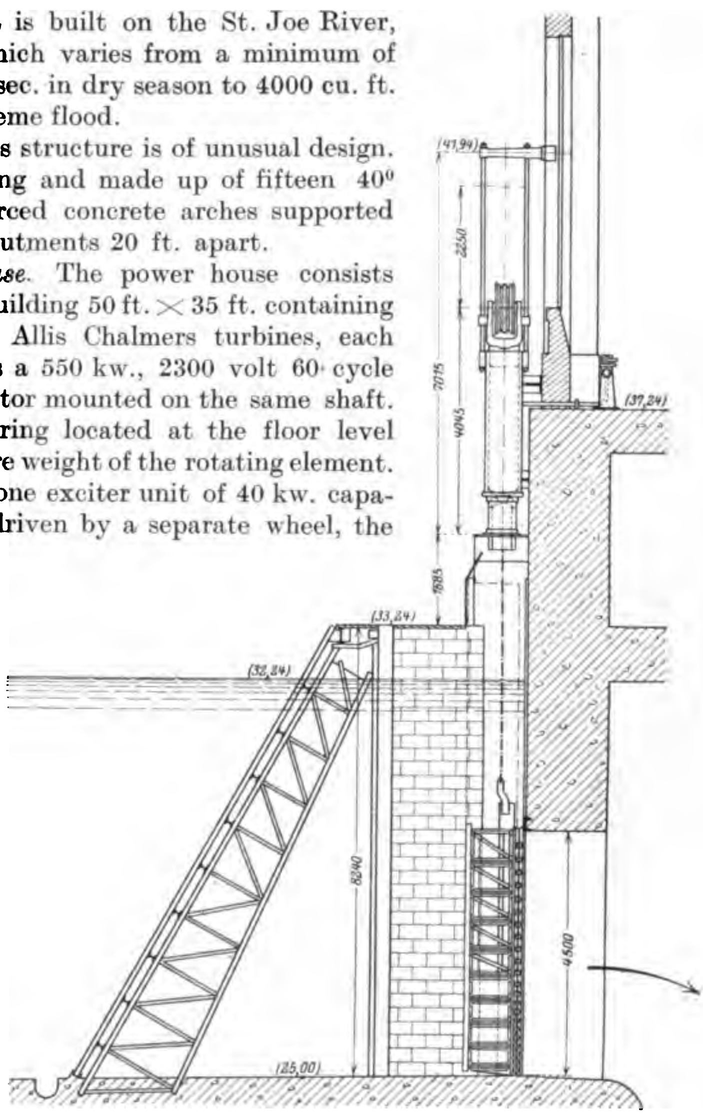


Fig. 310 Intake. Tuilière Plant.

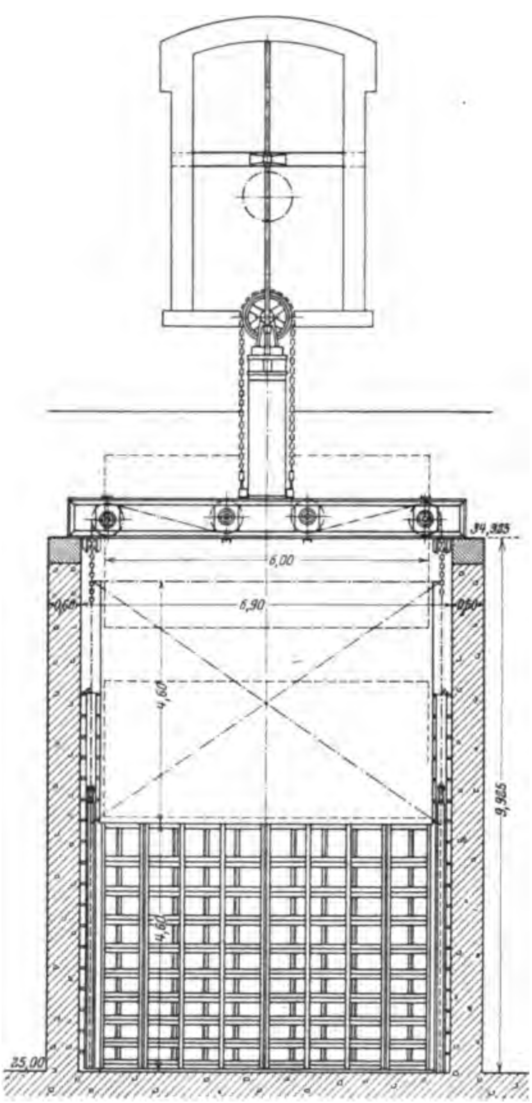


Fig. 311. Elevation of Intake Gate. Tuilière Plant.

latter being hand-regulated. The switchboard is located in a compartment behind the wheel pits. There are two groups of three 200 kw. transformers stepping the current up to a tension of 23,000 volts for transmission to the city of Sturgis and other points such as Centerville, Meaden, etc. The transmission line is about 20 miles long.

The design of this plant comprises several novel features, the most notable being the structure of the dam, and is a most economical scheme for a low head plant, (Fig. 314).

180. Menominee & Marinette Hydroelectric Development.

(Head 27 ft.)

This plant is located on the Menominee River, 20 miles above the cities of Menominee, Michigan, and Marinette, Wisconsin. The drainage area of the river above the dam is about 3700 sq. miles. The minimum

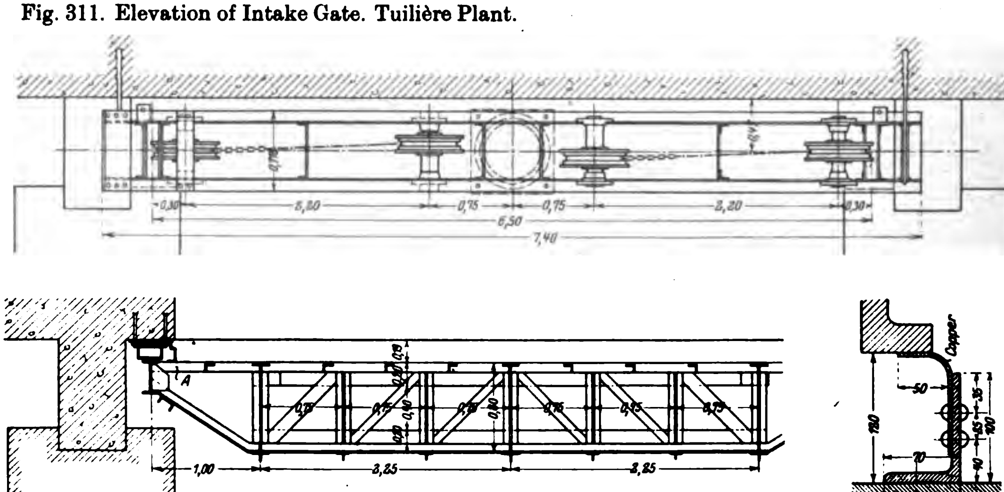


Fig. 312. Details of Gate. Tuilière Plant.

recorded flow was 2200 sec. ft. and the maximum estimated flow 12,000 cu. ft. per sec.

Dam. The dam is made of concrete and consists of two sections: the spillway 342 ft. long and 25 ft. high, and the remaining section consisting of a sluice gate system. The gates are 8 in number and of the type called the tainter gate. They are chiefly of wood as illustrated in fig. 316, and are raised by hand.

Canal. The canal is 3500 ft. long, 118.5 ft. wide at the bottom and the slope of the banks is 3 to 1.

Power house. The station is built of concrete, 120 ft. long and 32 ft. wide, fig. 315. The concrete flumes for the main turbines are 46 $\times$ 30 ft. in plan. In the center are two 46 $\times$ 8.5 ft. flumes for the exciter units. Two turbine units have been installed so far. Each consists of two pairs of 44-inch horizontal wicket gate turbines coupled to a 1100-kw, 2200-volt, 60-cycle, 3-phase generator with a speed of 150 r. p. m. Each exciter unit is coupled to a pair of 22-inch horizontal turbines having a speed of 350 r. p. m.

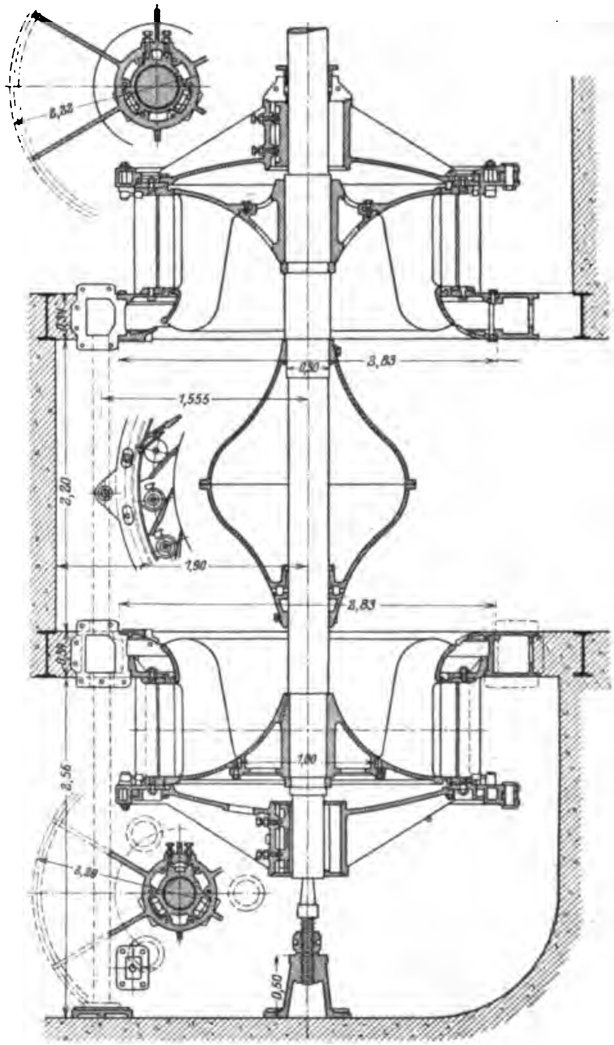


Fig. 313. Details of Turbine. Tuilière Plant.

Step-up transformers, which are located in a special transformer room, raise the voltage to 33,000 volts, and the energy is transmitted to the cities of Menominee and Marinette to supply them with light, power and street railway service. A typical load curve is shown in fig. 317.

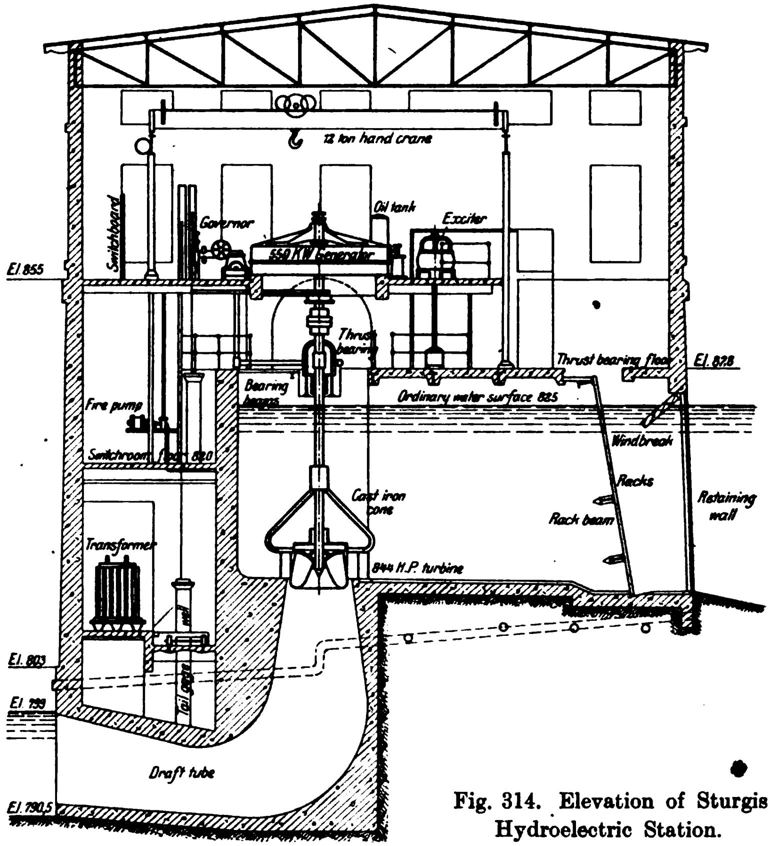


Fig. 314. Elevation of Sturgis Hydroelectric Station.

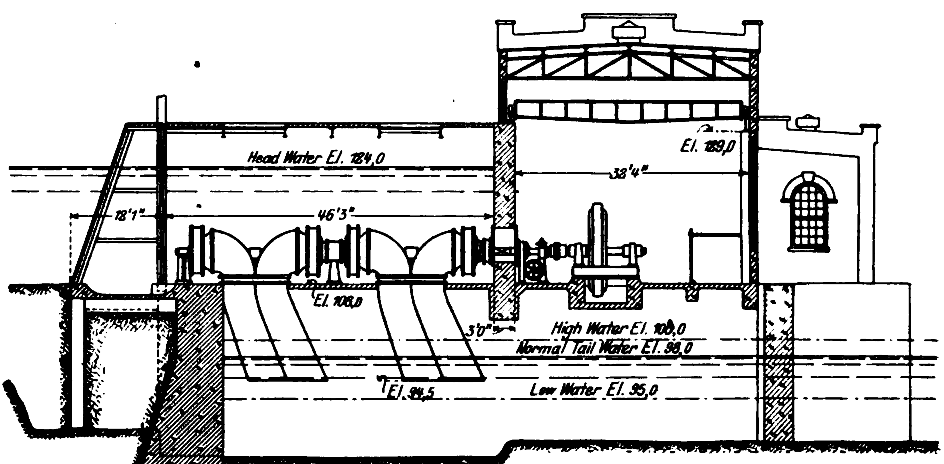


Fig. 315. Section through Main Generating Unit. Menominee and Marinette Station.

181. Johnsonville Development. New York.
(Head 24 to 35 ft.)

This plant has been built five miles upstream from the Schaghticoke plant, both using the waters of the Hoosic river. The drainage area of the river above Johnsonville is about 550 square miles, and the flow varies from 100 cu. ft. per sec. to 17,000 cu. ft. per sec. (See § 198.)

Dam. The dam is of the well known ogee type, running at right angles to the direction of flow, 530 ft. of spillway length, with a maximum height of 40 ft. and a maximum width at the base of 30 ft. Expansion joints have been provided every 50 ft. This dam ends in a masonry abutment. From this abutment which is curved in plan in order to deflect the waters, a concrete core-wall has been constructed running to the high ground. The height of this core-wall is 25 ft. at the

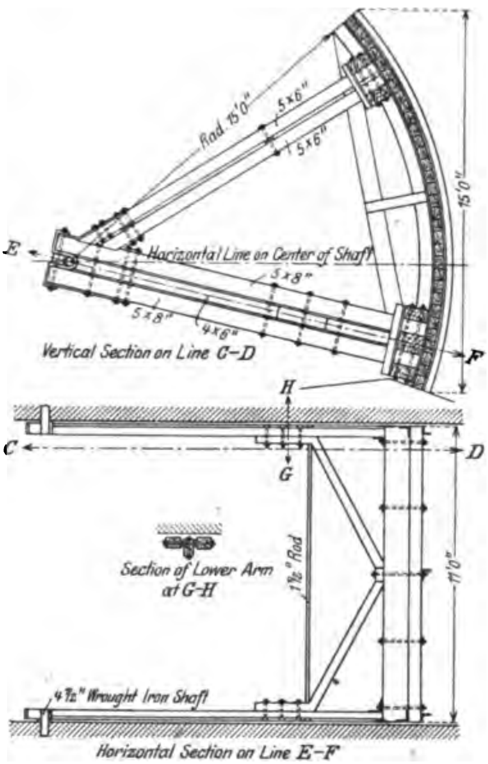


Fig. 316. Plan and Section of Wooden Tainter Gates.

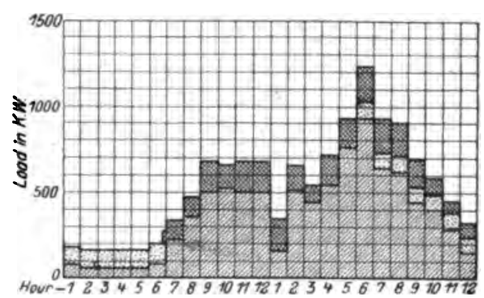


Fig. 317.
Load Curve for Nov. 30, 1910.

Menominee & Marinette Hydroelectric Development.

maximum point, and is covered by an earth embankment. Flash-boards as for the Schaghticoke dam are also provided, allowing a rise of 3 feet in the reservoir level. At the north shore end, 4 sluice gates 6 ft. $\times$ 9 ft. are located, allowing for a flood discharge of 8000 cu. ft. per sec. An ice chute is located between these sluice gates and the power house.

Intakes. The forebay is separated from the stream by a curtain wall about 160 ft. long, under which are three submerged openings. The top of this wall projects 3 ft. above high water. The wall is intended to deflect floating debris and ice to the ice chute. The water passes from the forebay through screens and gates enters two flumes, 20.5 ft. wide, 27.5 ft. long, the walls of which are of reinforced concrete.

Water wheels. A 57-inch double runner reaction turbine of the horizontal Francis type, is located in each flume. They develop 3000 HP under a head of 35 ft. and when the latter is reduced to 24 ft., the output is 1750 HP per wheel, the speed of 150 r. p. m. remaining constant.

Power house. This measures 37′-8″ by 83′-4″ in plan (fig. 318 and 319). The 1800 kw generators are direct-connected to the turbines by a shaft passing through a steel diaphragm. The generators are located in the south end, while two concrete platforms and the main generator floor at the north end accomodate the switchboard, transformers and other apparatus. The 3-phase current is generated at 4400 volts, 40 cycles, and each generator has a direct-connected exciter generating 110 volt direct

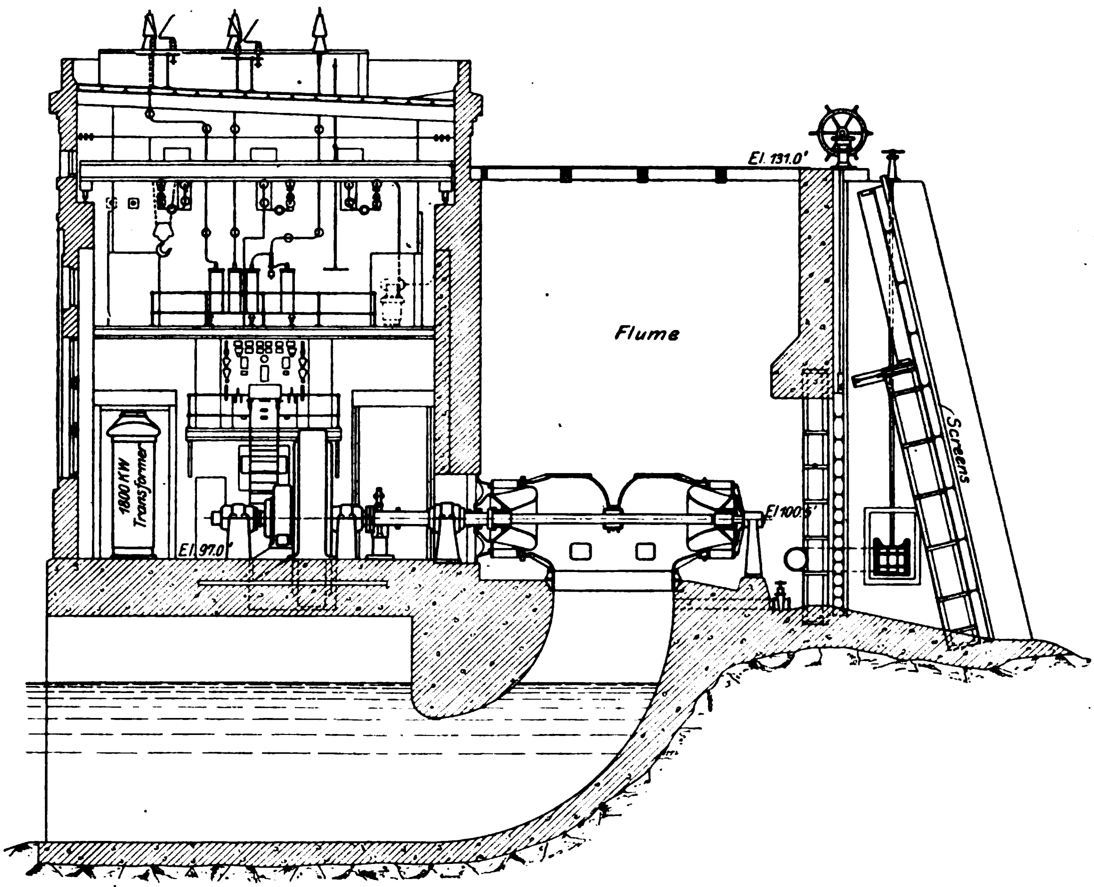


Fig. 318. Sectional view of Johnsonville Power House and Flume.

current. There are two 40 cycle 1800 kw, 4400/32,000 volt transformers transmitting the current to Schaghticoke. At the latter plant the transmission is tied in with the Schenectady circuits.

Transmission line. The transmission line is 5.8 miles long and composed of one single circuit supported by steel towers. These are 54 in number. The conductors are No. 2 B. & S. solid copper wires spaced 5 ft. 6 in. apart.

182. Mississippi River Hydroelectric Plant at Keokuk, Iowa[1]).

(Head 21 to 35 ft.)

The largest power plant in the world is at present in operation at Keokuk, Ia. Energy is furnished by the Des Moines Rapids of the Mississippi River, 140 miles above St. Louis. The initial installation is rated at 100,000 HP to be increased to 200,000 HP later on and finally 300,000 HP which is the full power of the river at this point.

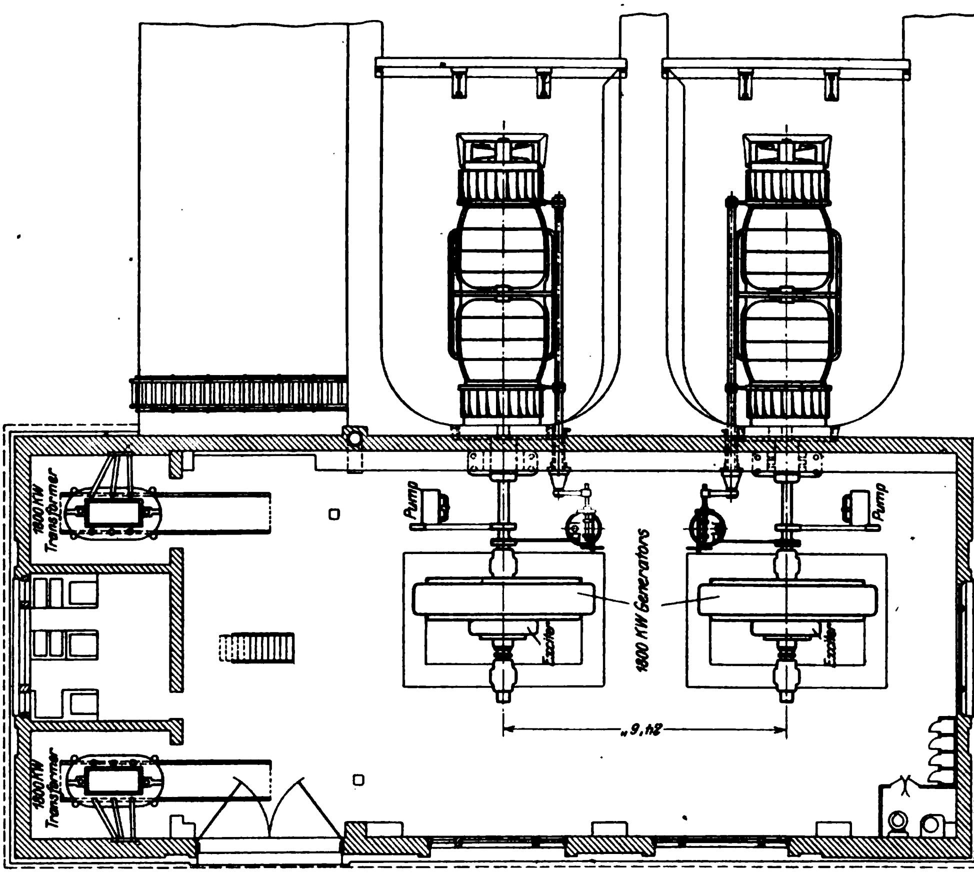


Fig. 319. Plan of Power House and Flumes showing the Location of Turbines, Generators and Governors. Johnsonville Plant.

For a distance of 12 miles above Keokuk, the Mississippi River is marked by shallow rapids, and in this distance undergoes a total fall of some 12 feet. The Government has constructed the existing canal and locks as an aid to navigation around these rapids. The general scheme involves the construction of a concrete dam at the site of lock No. 3 which will raise the level of the river by about 30 ft., submerging the rapids and creating an artificial lake extending 40 miles up stream, and having an average width of one mile. A single lift lock and a dry dock will be built in conjunction

[1]) The author in "La Technique Moderne", Paris, March 15th 1913.

with the dam. Fig. 320 is a sketch showing the dam structure and the power house as they appear to-day.

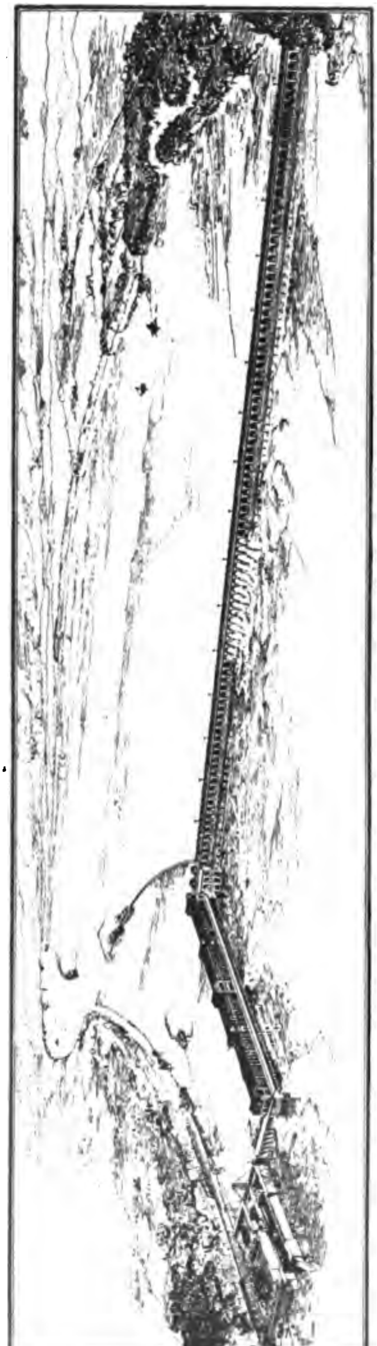


Fig. 320. Power Plant and Dam of the Mississippi River Power Company.

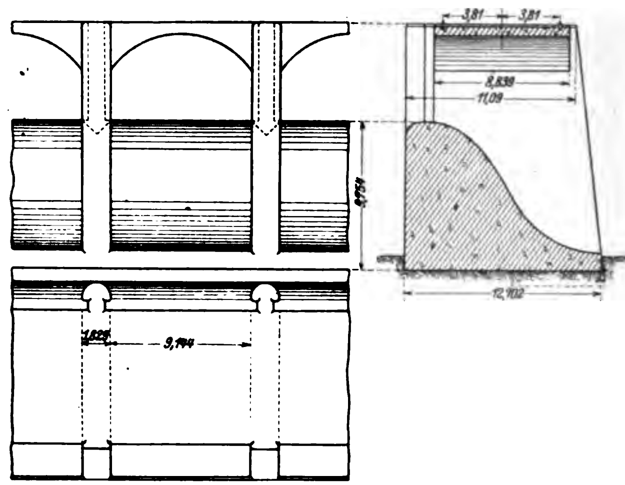


Fig. 321. Details of Spillway, Mississippi Development.

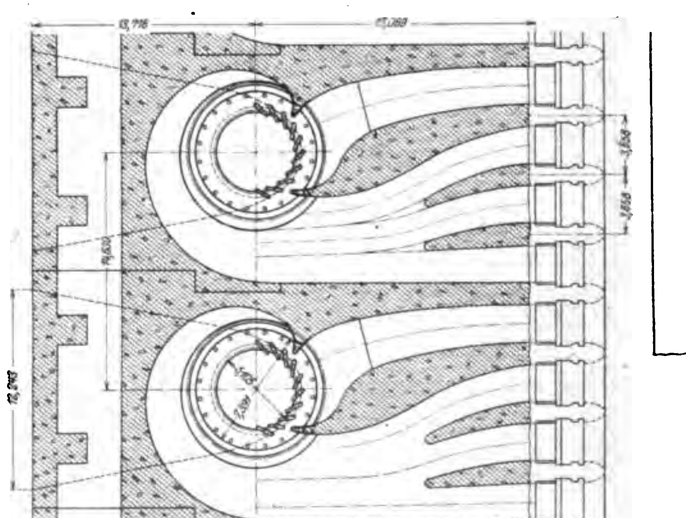


Fig. 322. Horizontal Cross-Section through Wheel Pit. Mississippi Development.

The dam has a total length of 4,700 ft. between abutments. The spillway section extends 4,400 ft. from the Illinois abutment. From the west end of the spillway, and at right angles to it, the power house extends

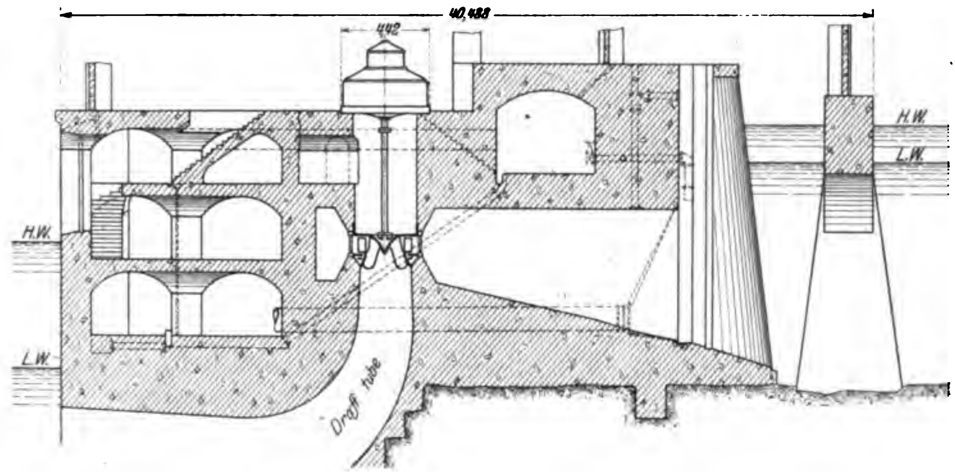


Fig. 323. Cross-Section through Exciter Unit. Mississippi Development.

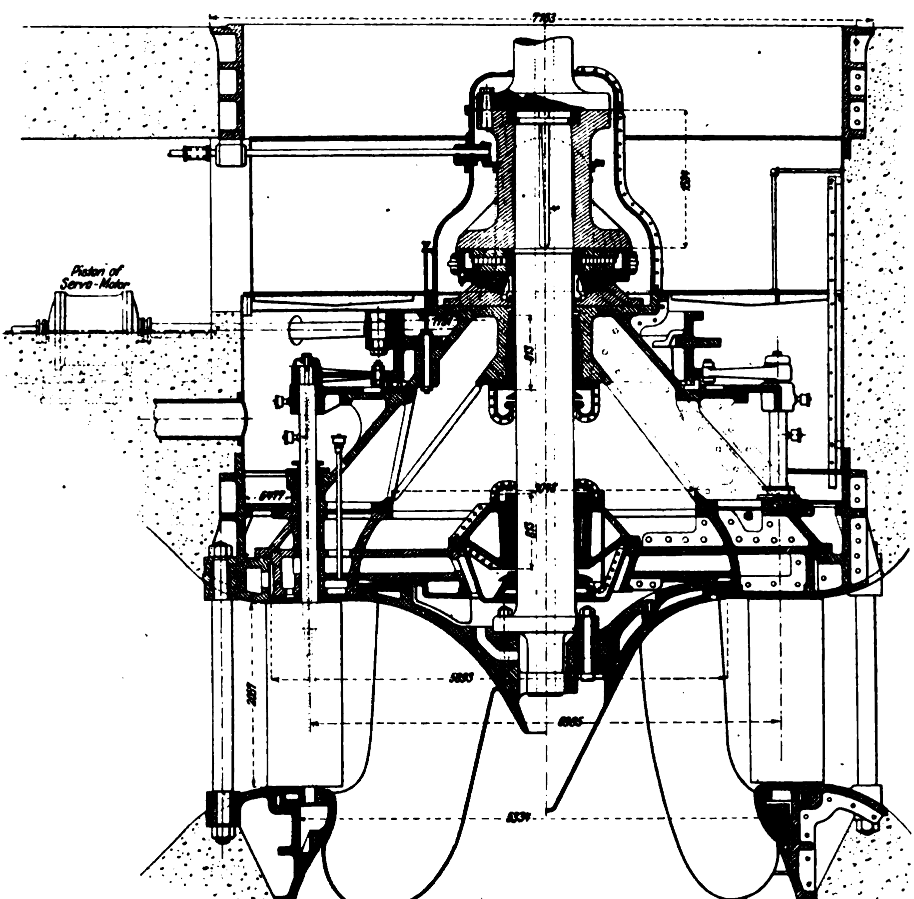


Fig. 324. Details of 10,000 HP Turbine. Mississippi Development.

1400 ft. down stream paralleling the Iowa shore at a distance of some 300 feet.

The dry dock basin and ship lock, 160 ft. × 450 ft., extends from the lower end of the station to the adjacent Iowa shore.

The principal spillway section of the dam, see fig. 321, contains 116 steel floodgates, each 30 ft. wide and 11 ft. high, is 4,400 ft. long 37 ft. high and 43 ft. wide at the base. The steel flood-gates are supported on concrete piers, 8 ft. thick and 29 ft. wide, built into and forming part of the main dam structure. These piers are also carry an arched passenger bridge from which the gates are operated by motor driven cab-hoists. The head available at the turbine shaft will vary from 21 to 35 feet, the flow of the river varying from 200,000 cu. ft. per second to 375,000 cu. ft. per second at flood times.

The power house is 1400 ft. long and 123 ft. wide and 133 ft. in height from foundations to roof. It contains 30 generators with the accessory transformer and switching apparatus. The generators consist of 5000 kw. units. The turbines illustrated in fig. 324, are of the vertical shaft type, the double runners being submerged in the wheel pit.

An arched concrete fender extends 2800 ft. upstream in order to protect the intakes against floating ice and debris.

The first transmission line built is a 110,000 volt 25 cycle line, carrying the energy to St. Louis. This is supported by steel towers provided with the suspension type insulator. Other lines will supply energy to neighbouring communities. It will be of interest to note that within a radius of 40 miles of the power house site, there is an aggregate population of 200,000 inhabitants; within 100 miles, 1,500,000; and within 150 miles 4,000,000 inhabitants.

The engineering work was done under the supervision of the well known engineer, Mr. Hugh L. Cooper of New York.

183. Plant of Winnepeg Electric Railway Company.

(Head 35 ft.)

This plant, which is illustrated in figure 325, is situated on the Winnipeg River, 65 miles from the city of the same name.

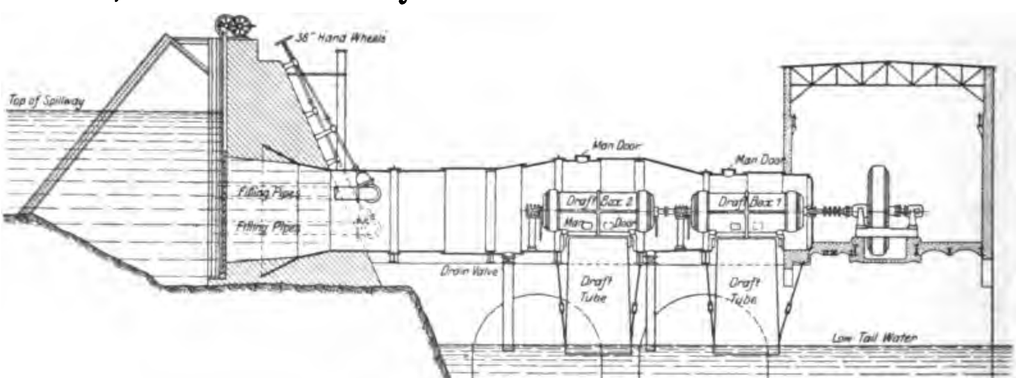


Fig. 325. Winnipeg, Plant, Manitoba.

Canal and dam. The canal is 120 ft. wide, and the water flowing in it has a clear depth of 8 ft. at normal low water. It is 8 miles long, with a slope of 5 ft. per mile. The dam is built at a point where there is a natural fall in the river bed.

Power house. The water wheels are of the Mc Cormick horizontal shaft type, and they are regulated by Lombard governors. These wheels are direct connected to four 1000 kw. and five 2000 kw. revolving field 2300 volt, 60 cycle three-phase generators. There are, besides, four exciters of a total capacity of 550 kw., delivering direct current at 125 volts. Two of these are coupled to turbines, two are coupled to three phase 2300 volt induction motors. There are 15 transformers by means of which the voltage is stepped up to 60,000 volts and the current is transmitted to the substation at Winnipeg, 65 miles distant.

184. Cedar Falls Plant, Wisconsin.

(Head 48 ft.)

The Chippewa Railway Light and Power Company's station of 10,000 kw is located on the Red Cedar River, 6 miles north of Menomie. The flow of the Red Cedar River is regulated by a system of 13 lakes, extending about 80 miles above Cedar Falls, and their storage capacity amounts to about 6,000,000 cu. ft. of water. The flow of the river itself varies from a minimum of 1500 cu. ft. per sec. to a maximum of 7500 cu. ft. per sec.

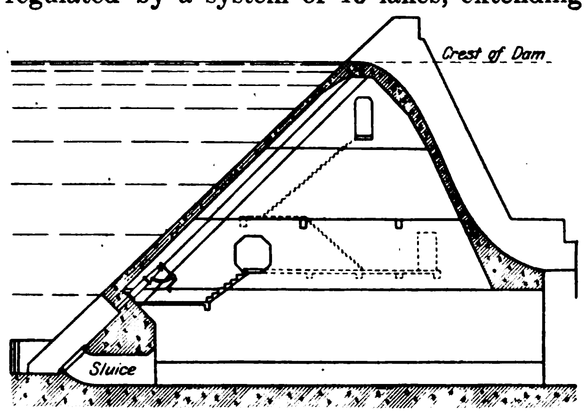


Fig. 326. Cross-Section of Hollow Concrete Dam at Sluice-Gate Bay. Cedar Falls Plant.

Dam. This is a reinforced concrete structure of the Ambursen type, 502 ft. in length, built of 28 bays, each of which is approximately 18 ft. wide. The spillway section is 252 ft. long and carried on 19 reinforced concrete buttresses, each 3 ft. thick at the base and tapering to 14 in. thick at the crest slab. The average width of the dam at the base formed by the footings of these buttresses is 80 ft. The upstream deck which is supported at a 45° angle is made up of reinforced concrete slabs 30 in. thick at the bottom and

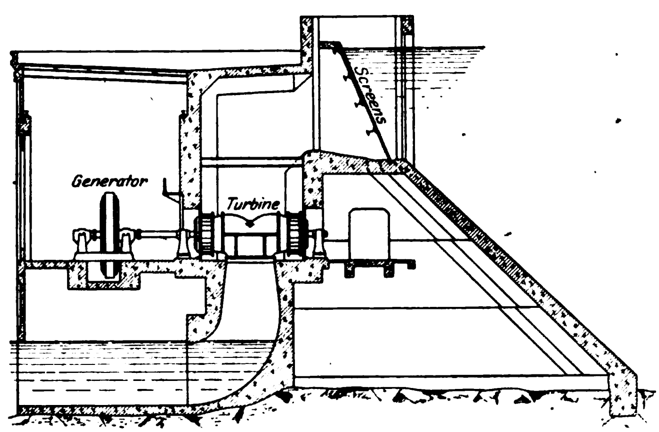


Fig. 327. Sectional Elevation of Cedar Falls Power House.

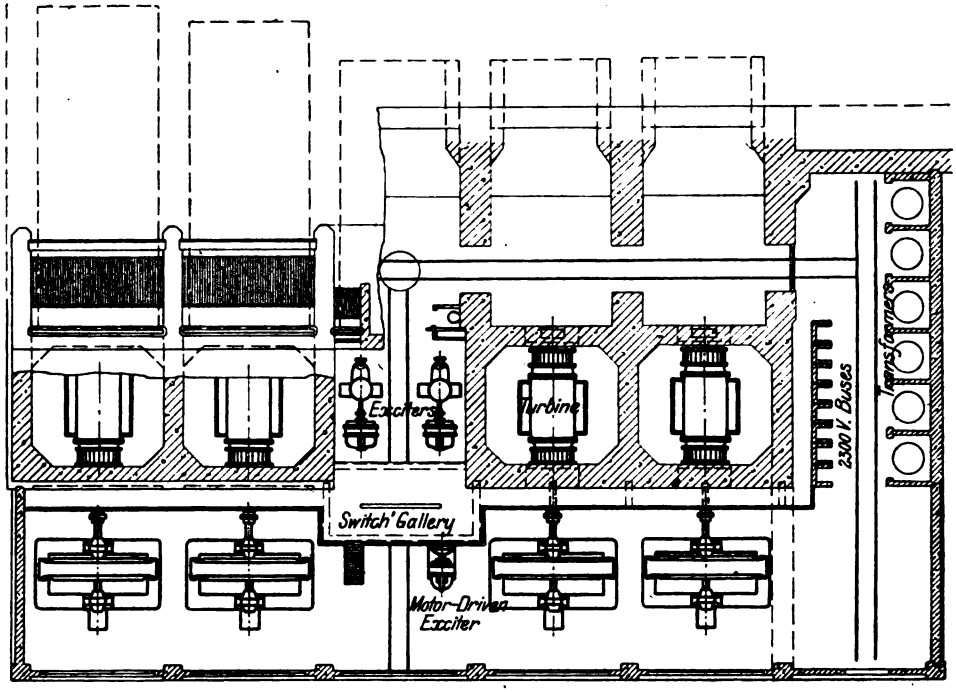


Fig. 328. Plan of Power House. Cedar Falls Plant.

12 in. thick at the top. The four spillway bays nearest the power house are provided with sluice gates discharging from the bottom of the forebay through 7 ft. × 14 ft. openings controlled by hand-operated gates, and operated from within the dam chambers (fig. 326). The crest of the spillway is provided with flash boards of the self leveling type, raising the level of the pond 2 ft.

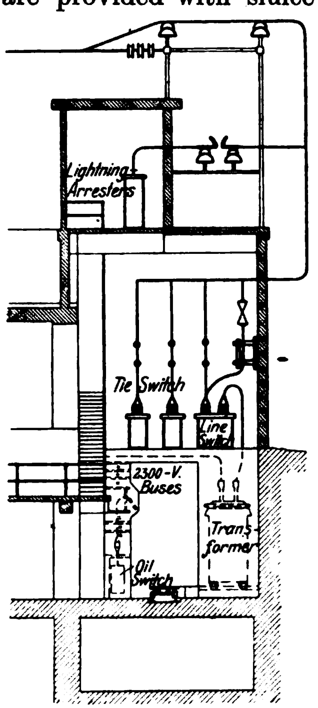


Fig. 329. Cross-Section through High-Tension Gallery.

Power house. The building (fig. 327 and 328) is a molded concrete structure, 138 ft. in length and 55 ft. in width. It accomodates four 2500 kw. turbo generator sets, each of 2500 kw. capacity, and two turbine driven exciter units. The transformer and high tension structure is located on the eastern end and contains the oil switches, lightning arresters and other apparatus. The switch board is mounted on a gallery overlooking the generator floor.

Turbines. The water passes through screens from the forebay into the rectangular penstocks down to the water wheels. The latter have 52.5 in. runners and are of a capacity of 3600 HP at 48 ft. head. The outer ends of the turbine shaft are carried in oil bearings, and are accessible for inspection. The down stream

bulkhead of the dam structure forms one wall of the turbine chambers, the rear bearing of the turbine shaft being reached by a passage beneath the deck of the dam. All of the turbines are controlled by Allis-Chalmers oil governors.

Generators. Each water-wheel is direct-connected to a 2300 volt 60 cycle alternator, generating 3-phase current. There are 3 exciter units, two water-wheel driven, and one motor-generator set. The turbine units are 100 kw. 125 volt direct current generators, running at 600 r. p. m.

The current from the generators is stepped up to 33,000 volts by means of oil-insulated, water-cooled, transformers, of which there are two banks of three 1666 kilovolt-ampere units.

Transmission line. This plant operates in parallel with the 1200 kw. plant of the same Company at Menomie, the 710 kw. plant and 300 kw. steam relay at Eau Claire and the 1000 HP plant at Chippewa Falls. These are interconnected by a 33,000 volt transmission system, which is extended 40 miles south of Menomie. The transmission line is a wooden pole line, and is protected by an overhead ground wire.

185. McCall Ferry Hydroelectric Development.

(Head 53 ft.)

The Susquehanna River, from which the power is obtained, has a catchment area of 26,766 sq. miles above the dam.

Spillway. This has a total length of 2500 ft., is of the ogee type, 53 ft. high at the top and 65 ft. wide at the bottom. It is built of concrete, the total quantity deposited in the dam having been 174,000 cu. yds. Fig. 329.

Power house. The power house is located at an angle of 42 degrees with the axis of the dam, and is placed adjacent to the Lancaster shore. The tail race is protected from overflow of the Susquehanna River by means of fender walls. The forebay is protected against floating debris and ice by means of a rock-filled ramp about 550 ft. long and perpendicular to the shore. Submerged arches, extending from the rockfilled ramp and the junction of the power house and the dam protect the forebay. Two floating booms are also provided. The maximum velocity of the water through the forebay entrance is 1.5 ft. per sec. Further protection is obtained by 11 arches on which rest the outer walls of the power house (see fig. 331). Here another ice chute is provided, the first one taking care of the forebay. Before entering the wheel pit the water passes through steel racks 55 ft. long. From the space behind the racks the water enters by four openings 6 ft. by 16 ft., which lead to the wheel pit.

The building itself is a concrete structure 500 ft. long. It accomodates 10 main units spaced 40 ft. apart, and two exciter units. It is divided transversally into 3 rooms, the generator room 48 ft. wide, the screen and gate room 56 ft. 8 inches wide and a two-story transformer room, 50 ft. wide. The transformers are in the lower story, whereas the switching apparatus is located in the upper story. Each turbine unit, described in paragraph 99, has a capacity of 13,500 HP at 80% gate opening with 53 ft. head. The wheels are of the Francis type and regulation is effected by movable vanes in the distributors. Each machine is direct

connected to a 7500 kw. 11,000 volt, 25-cycle, 3-phase generator. The exciters are of 1000 kw. each, generating 250 volt direct current at 240 r. p. m.

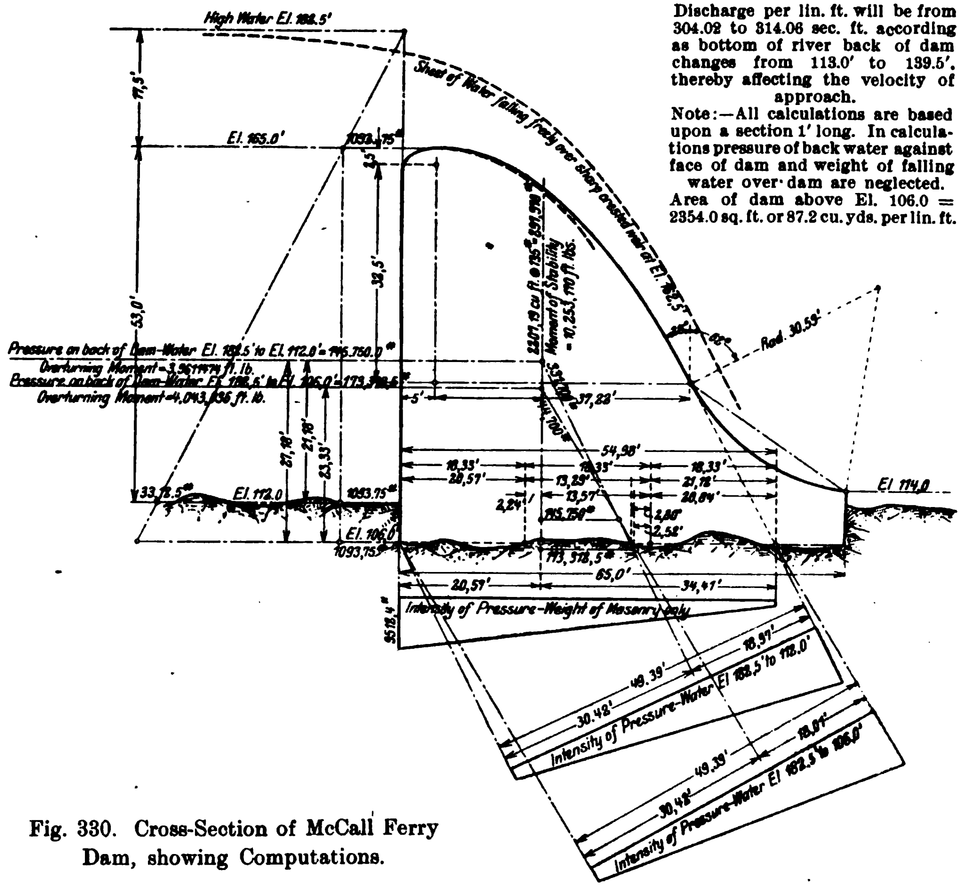


Fig. 330. Cross-Section of McCall Ferry Dam, showing Computations.

The McCall Ferry plant is located within 65 miles of Philadelphia, 40 miles from Harrisburg and Baltimore, 43 miles from Wilmington and 50 miles from Washington. It is estimated that an actual total amount of steam power of 750,000 HP is located within the commercial radius of this plant.

186. Ingeredsfors Plant, Sweden.

(Head 59 ft.)

The power station at Ingeredsfors deals with a normal discharge of 1630 cu. ft. of water per second at a head of 59 ft., and contains 3 generating sets each of 2750 HP supplying 3-phase current at 4000 volts 50 cycles, to three single phase 40,000 volt transformers. Energy is transmitted over a line 56 miles in length at 40,000 volts to the Mölndal sub-station, at the Gothenburg end of the line, where about 8000 HP are transformed to suitable pressures for distribution among the many cotton and paper mills and other factories in the Gothenburg suburbs of Mölndal, Krokslätt, Garda, etc. At the further sub-stations of Krokslatt and Garda in Gothen-

burg City, the pressure of the 3-phase current from the Mölndal transformer station is reduced from 10,000 volts to 210 volts and 400 volts and distributed by means of overhead cables. At the Varberg transformer station, of 2400 kw. rating, the line pressure is reduced to 4000 volts and

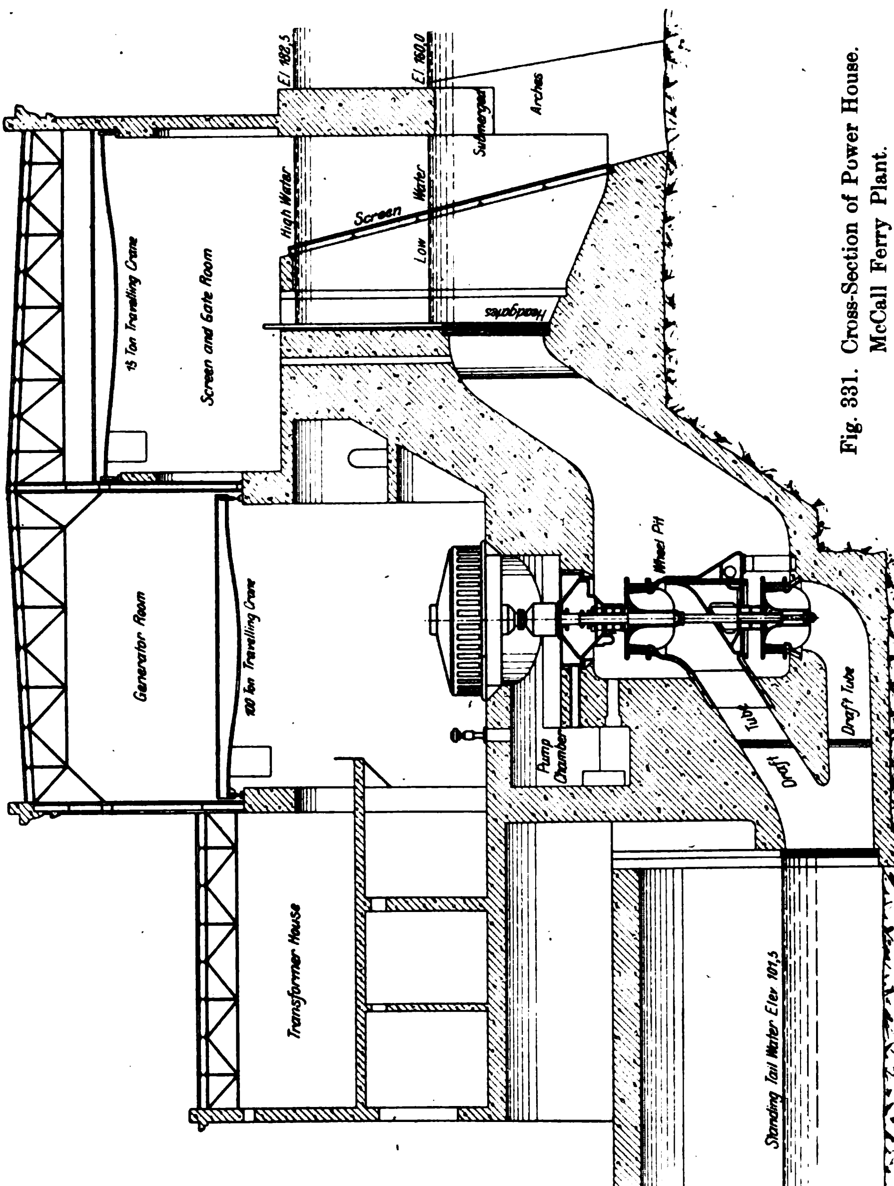


Fig. 331. Cross-Section of Power House.
McCall Ferry Plant.

the energy led to the sub-station in the town through two underground cables. There the pressure is reduced to 800 volts to supply energy to the motor net-work, and a storage battery is installed. The Varberg electricity works of which this substation forms a part, also contain a complete 2750 HP steam turbo-generator plant. This is intended to act as a reserve or stand-

by in case of disturbances or low water at Ingeredfors, in which case the transformers would have to step up the e. m. f. to 40,000 volts to assist the main line.

187. Sioux Falls Development, South Dakota.

(Head 60 ft.)

Dam and intake. The motive power of this plant is derived from the Sioux River, South Dakota, its average flow being 250 cu. ft. per sec.

The dam has a maximum height of 8 ft. with approximately 600 ft. of spillway. The intake is provided with a submerged arch which prevents ice and other floating matter to enter the forebay and clog the screens. From this intake, a penstock 7 ft. in diameter and 680 ft. in length, with a plate

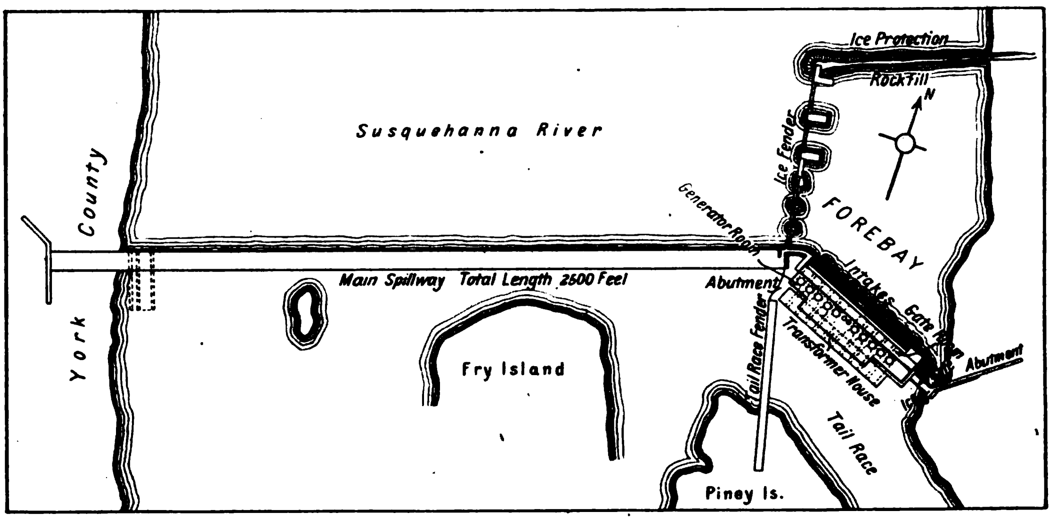


Fig. 332. McCall Ferry Development.

thickness of $\frac{1}{4}$ inch, carries the water to a surge tank situated about 25 ft. from the power house. The surge tank is 62 ft. high and 16 ft. in diameter, and serves for water regulation purposes. From this a steel pipe 9 ft. in diameter leads the water to the wheels in the power house.

Power house. The power station accomodates four 500 kw. turbo-generators, one 50 kw. motor driven exciter, a 50 kw. turbine driven exciter and two 150 kw. railway motor generator sets. The main generating units which were supplied by the Allis Chalmers Company, consist of 3-phase 60 cycles 2300 volt machines turning at 300 r. p. m. If future conditions so demand, the plant will be operated under an 80 ft. head. For this purpose provision is made for installing long draft tubes extending in long bends and supported by concrete piers. The plant, being located within the city limits, power is developed without transformer stations or transmission lines. The outgoing lines which supply current to operate the street railway system and other industries, consist of four 3-phase, 2300 volt lines, and six single phase 2300 volt lines.

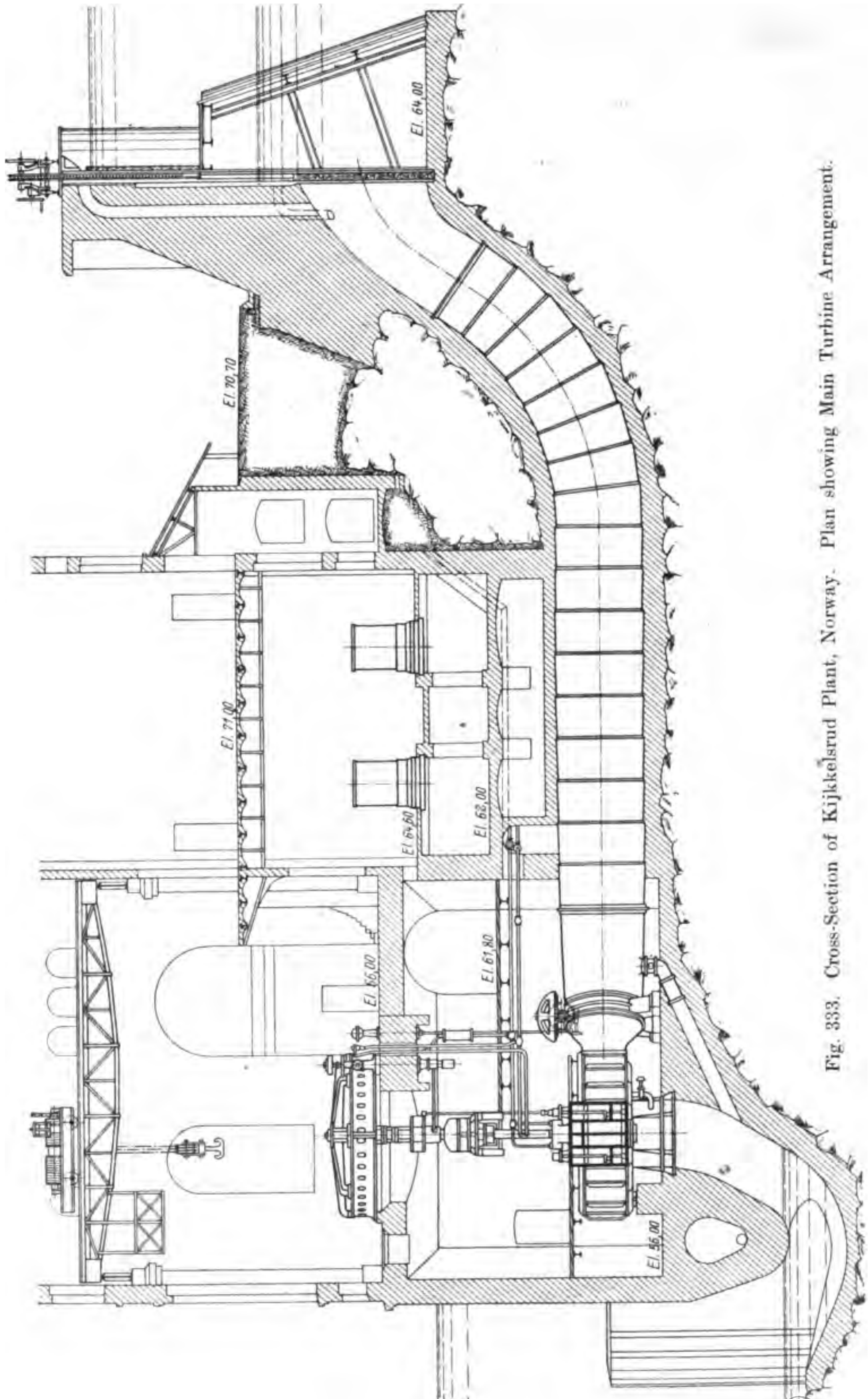


Fig. 333. Cross-Section of Kijkkelsrud Plant, Norway. Plan showing Main Turbine Arrangement.

188. Kijkkelsrud Plant, Norway.
(Head 60 ft.)

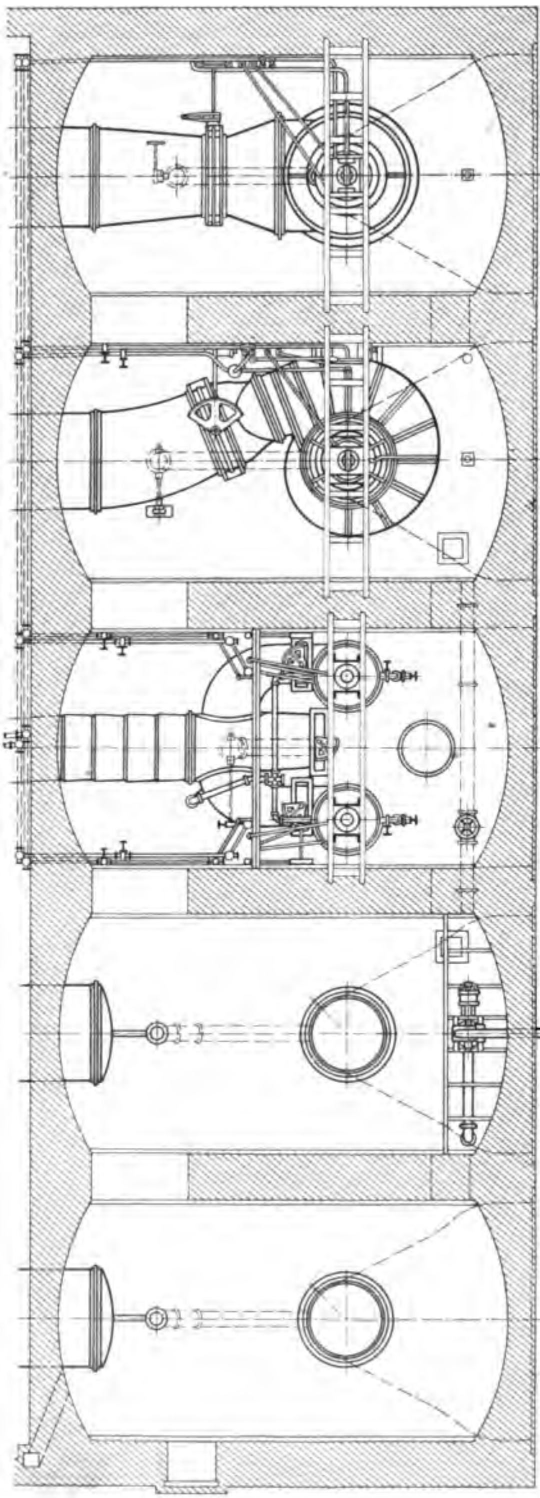


Fig. 334. Plan of Turbine Chambers, Kijkkelsrud Power Plant, Norway.

This hydroelectric development is located about 38 miles southeast of Christiana, the capital of Norway. The waters of the River Glommen are utilized for this purpose, and this river offers the advantage of many natural lakes as storage reservoirs. The drainage area is approximately 16,000 square miles.

Intakes. Water is carried to the forebay of the power house by means of a canal 2300 ft. long and 40 ft. deep. The canal intake consists of two sluice gates, each divided into two sections. They can be either hand or electrically operated. The forebay is 420 ft. long and is provided with 3 sluice gates to discharge anchor ice and sediments, a spillway located at one end is 330 ft. long and takes care of sudden flood waters and floating ice.

Penstocks. These are at right angles to the forebay, protected by screens and operated by gates from the top of the forebay wall. They are 9.8 ft. in diameter and imbedded into the concrete. Each is provided with a geared butterfly valve at the turbine casing entrance. There is one penstock for exciter, 6.5 ft. in diameter.

Power house. The building is 150 ft. long and 50 ft. wide. It accomodates 3 main turbo-generator sets, and two exciter sets. Two of the turbines are of the inclosed

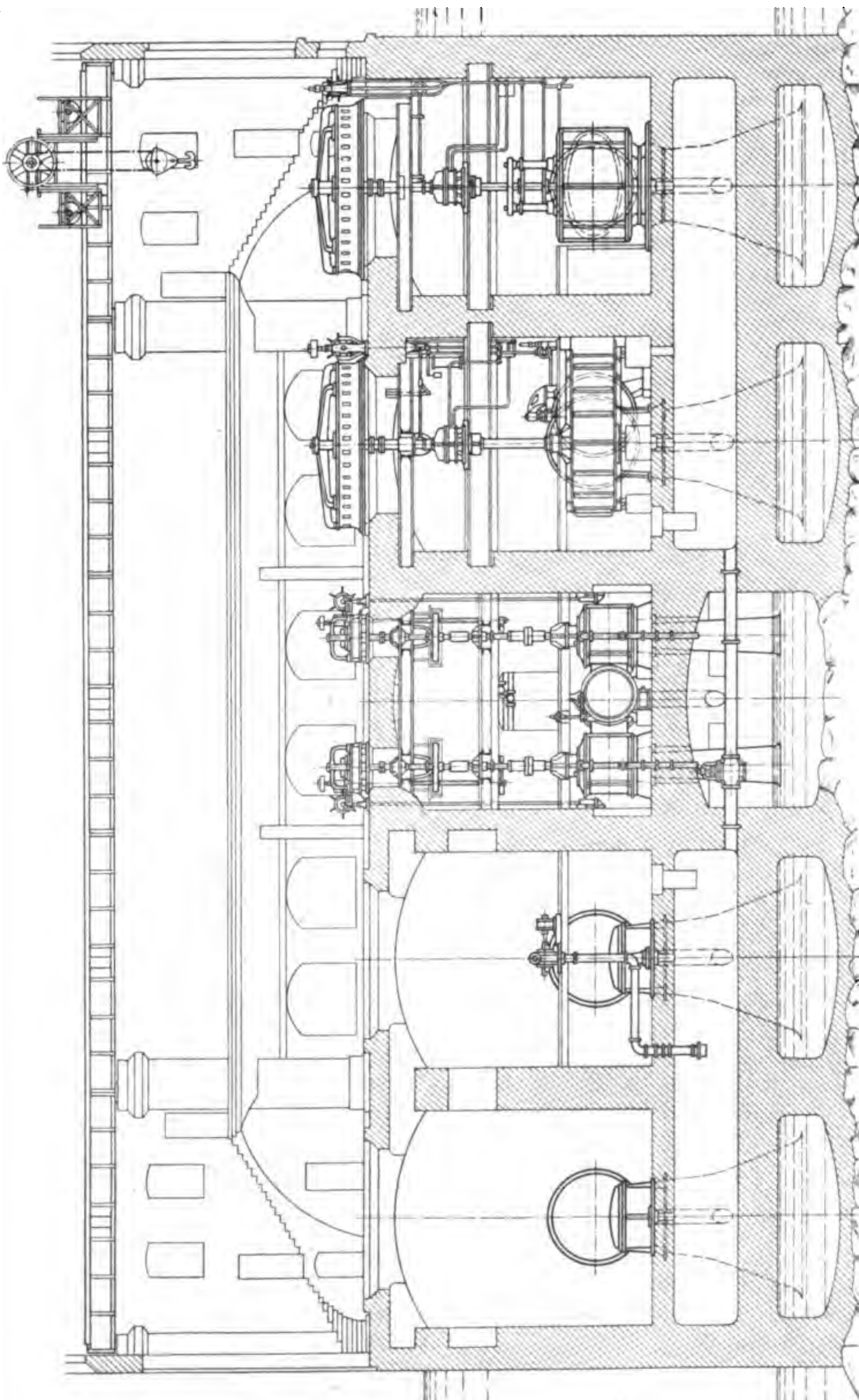


Fig. 335. Longitudinal Section of Kijkkelsrud Plant, Norway.

vertical Francis type, of 3000 HP capacity with a speed of 150 r. p. m. Their regulation is obtained by a governor hydraulically operated. The other two turbines are of the same type but have a capacity of 3750 HP. Each of the turbines drives a direct-connected generator of a capacity of 2000 kw. for the 3000 HP wheels and 2500 kw. for the 3750 HP wheels. Three-phase current is generated at 50,000 volts 50 cycles.

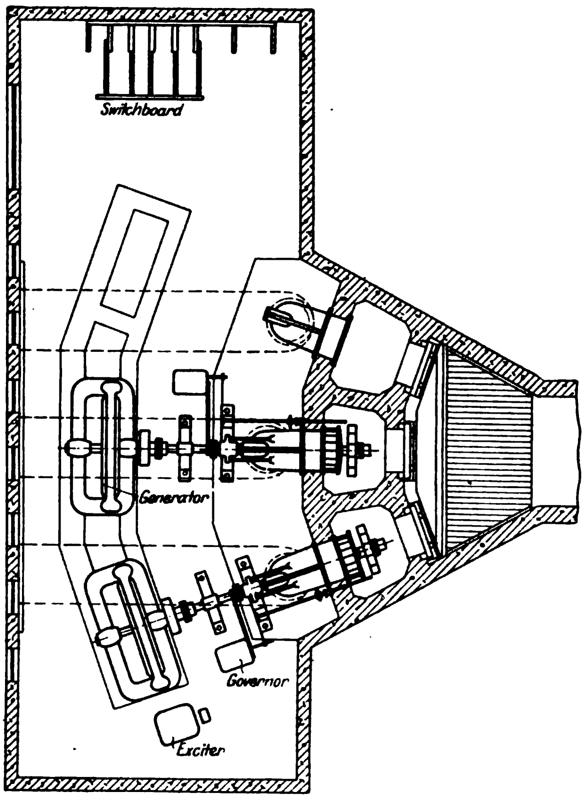


Fig. 336. Plan of Power-House Bulkhead and Flume, Erindale Power Company.

Three are 2 exciter sets, direct connected, of 182 kw. capacity and generating direct current at 115 volts.

The transformer and switching room is located behind the generator room; the switchboard is located on a gallery 16.5 ft. above the main floor. There are three transformers for each 2000 kw. generator, their capacity being 950 kw. each and their voltage 5000/20,000. They are of the aircooled oil type. The other two generators are connected to four 3-phase transformers of 2250 K.V.A. capacity. These are of the water-cooled oil type and wound for 5000/50,000 volt.

Transmission lines. The total length of the 20,000 volt transmission line is 53 miles, there being 7 sub-stations along the line, stepping the voltage down to 5000 volts. The conductors have a cross section area of 50 sq. m.m. for a distance of 38 miles, this area being reduced to 35 sq. m.m. for the remaining 15 miles.

The poles are made of wood, 40 ft. high and buried 6 ft. into the ground. The 50,000 volt line is used exclusively to transmit energy to Hafslund.

189. Erindale Power Company's Plant. Canada.

(Head 60 ft.)

This station is located at Erindale, on the Credit River, 4 miles from Ontario. The river has a minimum flow of 120 cu. ft. per sec.

Dam. A dam has been built across the river and is 700 ft. long. It comprises a 100 ft. spillway 34 ft. high the remainder being an earth embankment.

Tunnel. The water is conveyed to the power house by means of a tunnel 900 ft. long and 12.5 ft. in diameter.

Power house. The station contains two water wheels of the combined-impulse and reaction type, 50 inches in diameter, developing 840 HP each, with a speed of 200 r. p. m. They are direct connected to a 13,200-volt 60-cycle, 3-phase generator. (Fig. 336-337.)

Transmission lines. A single circuit of No. O Aluminium wire is carried on a wooden pole line. The potential is stepped down to 2200 volts, 2-phase, by means of T-connected transformers rated at 300 kw. each and located in a substation at West Toronto.

190. Marble Falls Plant. Texas.

(Head 70 ft.)

On the Colorado River, in central Texas, a system of three power developments is being contemplated with an aggregate output of 20,000 HP.

The Marble Falls plant, which forms part of the system is nearly completed and energy will be transmitted to Austin, Temple and San Antonio. This development comprises an immediate output of 1650 HP, which will be increased to 9650 HP later on. With the storage area of 116,000 acre ft. impounded by an 80 ft. dam across the San Saba River a minimum flow of 740 cu. ft. per sec. is assured.

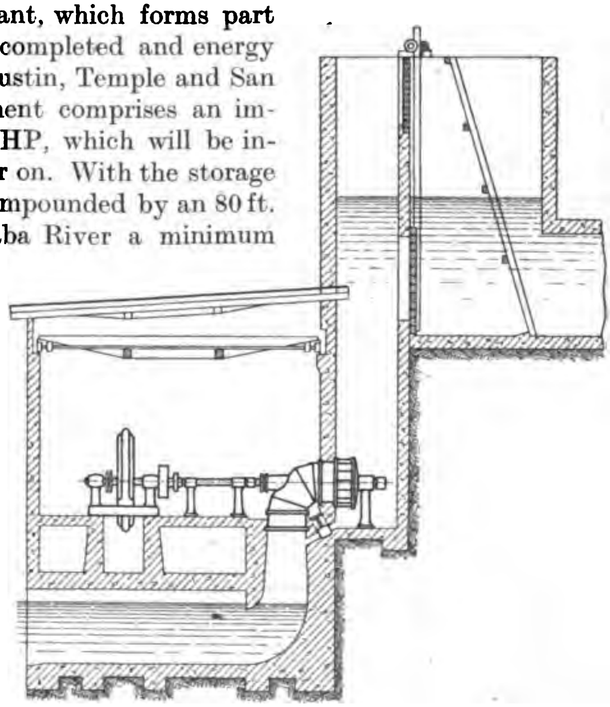


Fig. 337. Bulkhead Flume and Cross-Section of Power House. Erindale Power Company.

Dam. This structure Fig. 338 is of the concrete compartment type, 748 ft. long at the top and 700 ft. long at the base. The masonry spillway, 400 ft. long, rises to a height of 65 ft. Flash boards 5 ft. high are also provided. The buttresses of the spillway section are 10 ft. apart, and those in the power house section are spaced 14.5 ft. The 35 sluice gates, each 7.5 ft. $\times$ 8 ft., are built of 9 in. channels reinforced by beams and covered with $\frac{3}{8}$ in steel plate.

Power house. This building, which really forms part of the main dam, is built to a height above any possible high water (Fig. 338).

Three 550 HP Victor vertical water wheels, direct connected to 350 kw., 2300 volt, 60 cycle 3-phase alternators are installed. 2200 HP units will be placed later on. The power will be transmitted at 66,000 volts over a maximum distance of 130 miles.

191. Great Falls Power Plant.

(Head 72 ft.)

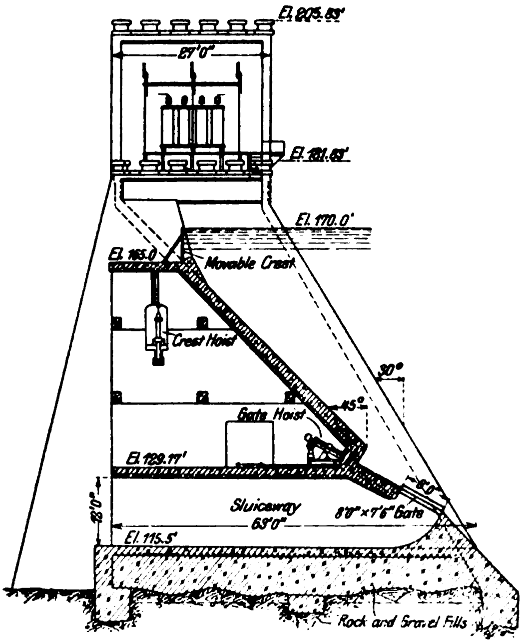


Fig. 338. Sluice Gate Section of Dam. Marble Falls Plant.

The Great Falls Power plant, which is owned by the Southern Power Company, derives its energy from the Catawba River, at a point called Great Falls, where a spillway dam has been constructed. The drainage area above these falls is 4200 sq. miles.

Spillway dam. The main spillway is 438.8 ft. long at the crest and has an average height of 30 ft. and a width at base of 41 ft. The spillway dam in the canal is similar in design, 521.2 ft. long at the crest and 36 ft. high with a width at the base of 37.75 ft. The main dam has a height of 90 feet and the width at the top is 8 ft. The upstream face is vertical and the downstream face is battened $1\frac{3}{4}$ to 1.

Power house. The station is 250 ft. long and 37 ft. wide. The switch and transformer house, which forms an adjoining two-story building is 85 ft. long and 75 ft. wide. There are eight main units, each consisting of a pair of horizontal turbines, with top inlet and central discharge. They have a capacity of 5200 HP, with a speed of 225 r. p. m. They are direct connected to 3000 kw. 2300 volt 60 cycles alternators. The exciters units are of 700 HP capacity each, making 450 r. p. m. and generating 400 kw. at 250 volts. The cross-section through the power house is illustrated in fig. 340.

The switchboard is located on a platform, and is composed of two transformer panels, two double circuit feeder panels and one station panel. There are besides two blank panels for future use.

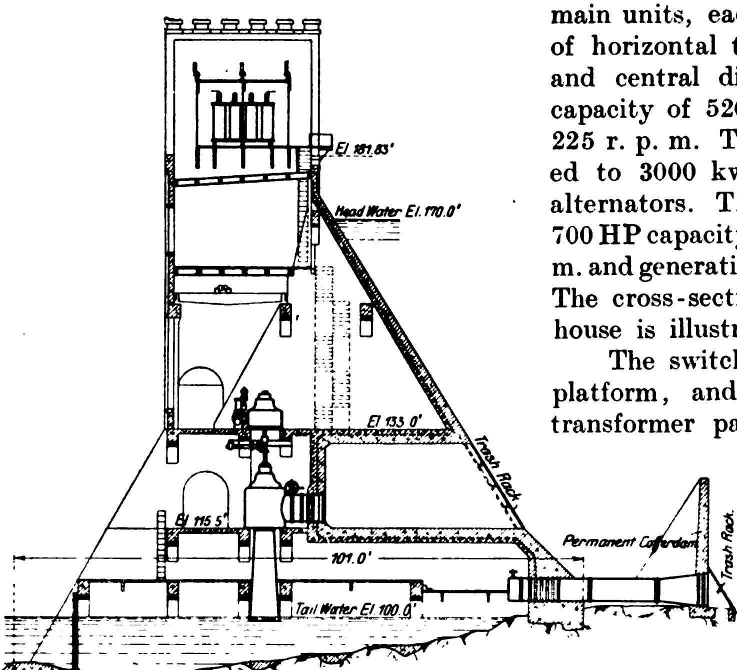


Fig. 339. Vertical Section through Power House. Marble Falls Plant.

The transformers, 12 in number are arranged in four banks of 3 each. Their capacity is 2000 kw. and

are of the oil-insulated water-cooled type. The ratio of transformation is 2300/44,000 volts.

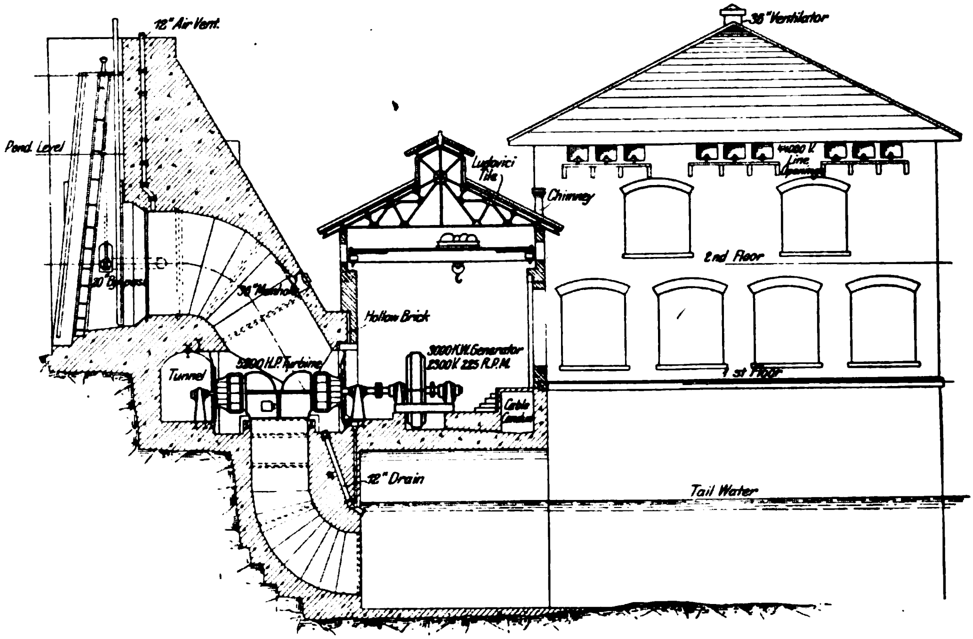


Fig. 340. Sectional Elevation of Great Falls Power Plant.

Transmission lines. Energy is transmitted from the Great Falls station to the Catawba sub-station by means of a three-phase circuit composed of six strand cable with hemp core equivalent to No. 000 and supported by galvanized two-circuit steel towers, which are spaced at approximately 420 ft. apart.

192. Hydroelectric Plant of Cervara. Italy.

(Head 75.5 ft.)

This plant has been constructed on the River Nera in order to furnish power to the electro-chemical works of Narni.

The discharge of the river is approximately 1400 cu. ft. per sec.

Intakes. The forebay consists of six intakes, 6.5 ft. in dia. each one with its corresponding steel gate admitting the water to an open canal of trapezoidal cross section, having a slope of 0.001, 1420 ft. long, 6.5 ft. deep and an average width of 39 ft. This canal ends at a settling basin, connecting

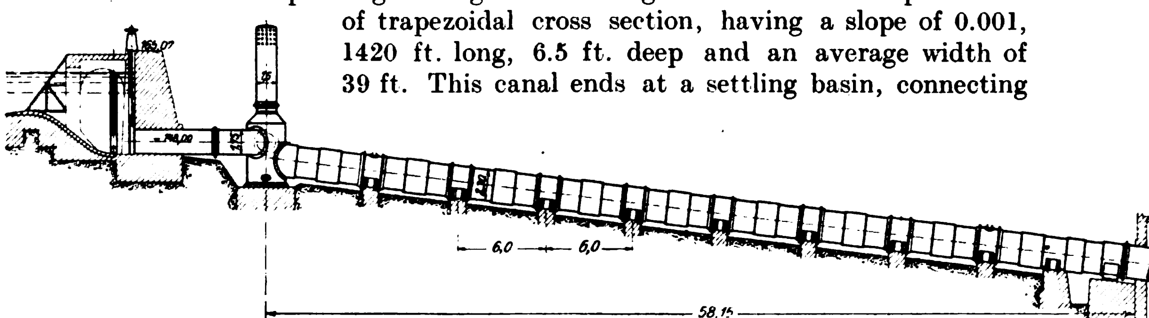


Fig. 341. Pressure Pipes. Cervara Station.

with it by means of 5 locks. A by-pass permits to shut off this basin in case repairs are to be made. The racks project nearly 6 ft. above water level.

Pressure pipes. There are two penstocks, 11.5 ft. in dia. and 1700 ft. long conducting the water to a pressure tank. In this tank there are twelve gates, 9 of which in groups of 3, feed two pressure pipes each; one pipe per unit of 2200 HP. In this manner the 9 gates command the 6 units; two other gates command two turbines of 1000 HP and the 12th gate corresponds to the exciter set. The main pressure pipes are of riveted Siemens Martin steel $\frac{5}{16}$ in. shell thickness. The exciter pipe has a diameter of 5.75 ft. An expansion joint is located on each pipe line (see fig. 341).

Power house. The generator room is 216.5 ft. long and 49.2 ft. wide; it accomodates 6 horizontal reaction turbines of 2200 HP, directly connected to 1900 kw. alternators. These as well as the two 865 kw. units, generator 3-phase current at 3750 volts 42 cycles and runs at 250 r. p. m. Two 75 kw. exciters furnish current of 125 volts at 500 r. p. m. by means of common bus-bars. There is a third exciter of 150 kw. capacity run by a 240 HP induction motor. The current generated by the alternators is stepped up to a tension of 27,000 volts by means of 1900 kw. star wound air-cooled transformers.

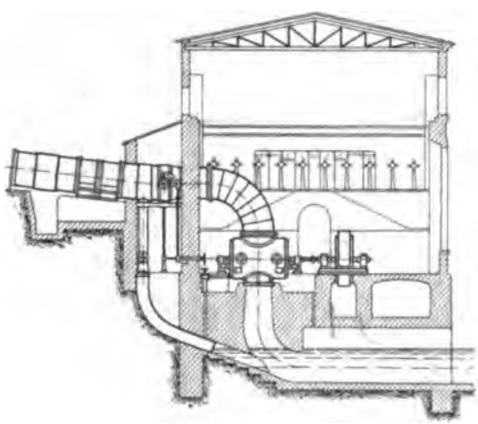


Fig. 342. Cross-Section of Power House. Cervara Station.

The switchboard, composed of 9 panels, on which are mounted the usual instruments, is situated on a platform which dominates the generator room.

Transmission line. The current is transmitted at 27,000 volts over two separate circuits eight and a half miles in length, and consists of 6 conductors No. 1 B. & S.

The insulators are mounted on steel towers spaced about 300 ft. apart. The minimum clearance between the conductors and the ground is 24 ft. In the electro-chemical works of Narni, there is located a sub-station in which the current is stepped down according to the needs of the plant.

193. High Falls Plant. Wisc.

(Head 85 ft.)

This plant was developed in order to supply the electric power to Green Bay, Wisc., and other adjoining towns. The Peshtigo River on which this plant is located has a length of some 90 miles with a fall of nearly a thousand feet. The river discharge varies from 300 cu. ft. per sec. to 700 cu. ft. per sec. at flood times.

Dam. The aggregate length of the dam is 4500 ft., and is made of a main concrete section extending over a length of nearly 900 ft., which is

tied to the shore by means of earthen wing walls. The height of the dam is 45 ft., and its base width 64 ft. Flood discharge is taken care of by means of a 180 ft. spillway of the ogee type. A log chute also is provided in accordance with the provision of the Wisconsin State law; its width is 8 ft. and its length 260 ft. The wing walls are of gravel and earth with a concrete core wall. Their width at the top is ten ft., the upstream slope is 1 to 3 and the down stream slope 1 to 2. The available head created by the dam is 85 ft., and the storage capacity 859,805,000 cu. ft.

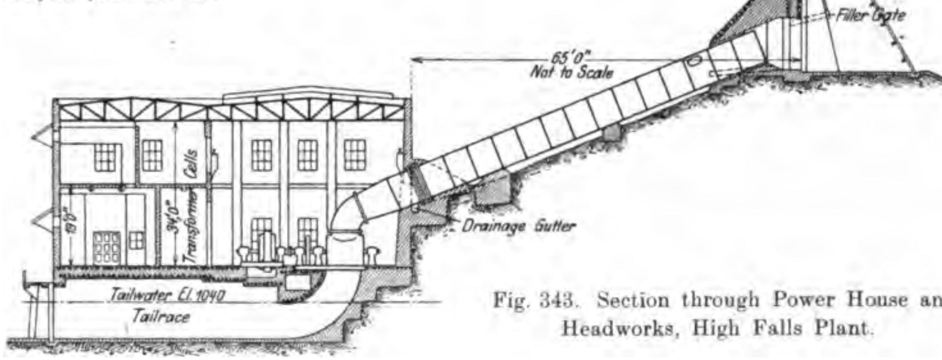


Fig. 343. Section through Power House and Headworks, High Falls Plant.

Intakes. The water is admitted to the pipe lines through double vertical lift gates, operated by electric motors. Flood gates are provided also, being 6 in number, with an opening 12 ft. × 12 ft. These are operated by a hand hoist mounted on a track and in such a way is made to handle any of the gates.

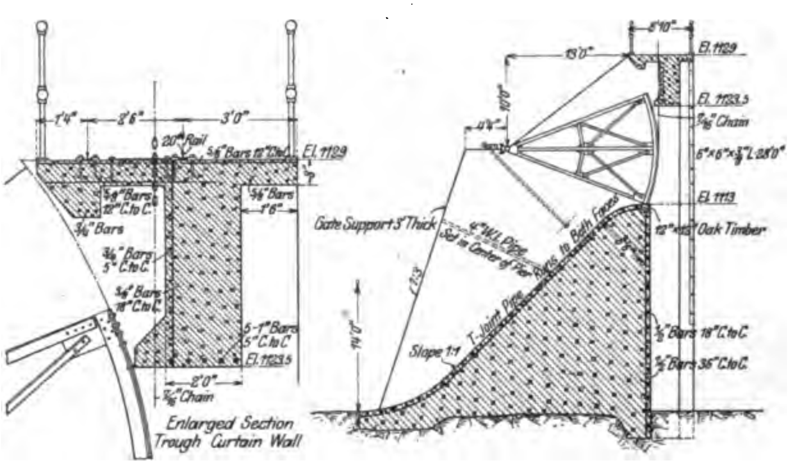


Fig. 344. Spillway Section at Tainter Gate. High Falls Plant.

Pipe lines. Five riveted steel pipes, 8 ft. in diameter and 80 ft. long, $\frac{3}{8}''$ plate, carry the water to the main turbines. Screens are provided, as well as filler gates, and the vent pipes are 10 inches in diameter. The exciter pipes are 3 ft. in diameter, and the intake design provides for 2 future units.

Turbines. The power house, see figure 343, is a two story structure, 136 ft. long and 83 ft. wide. It contains at present 5—1000 kw. turbine

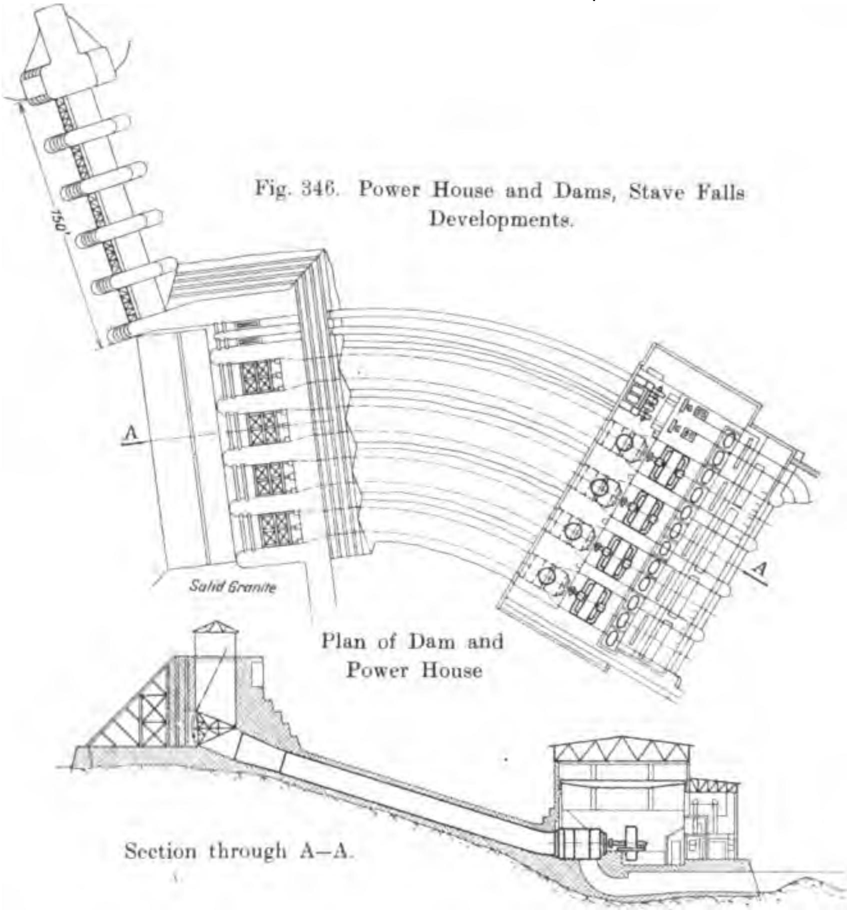


Fig. 346. Power House and Dams, Stave Falls Developments.

Plan of Dam and Power House

Section through A—A.

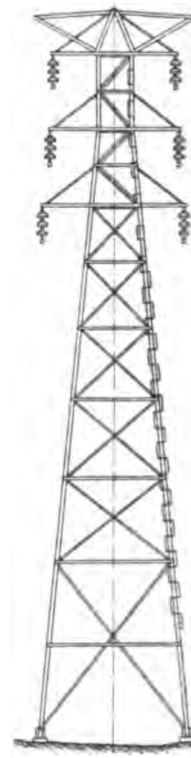


Fig. 345. High Falls Transmission Line Tower.

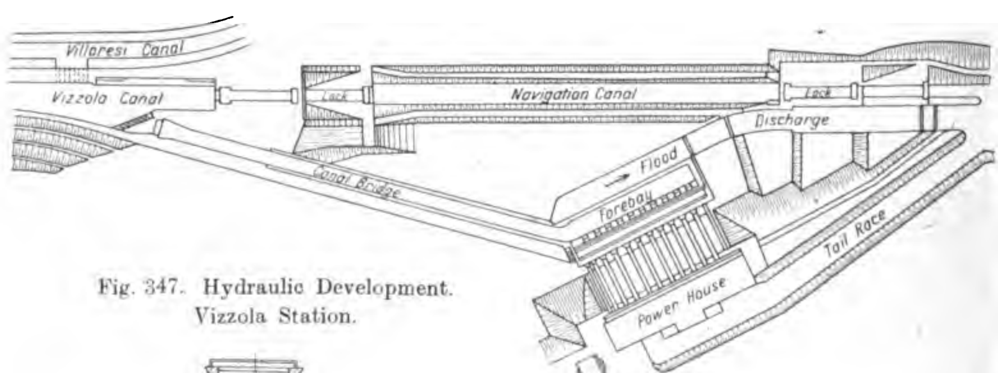


Fig. 347. Hydraulic Development. Vizzola Station.

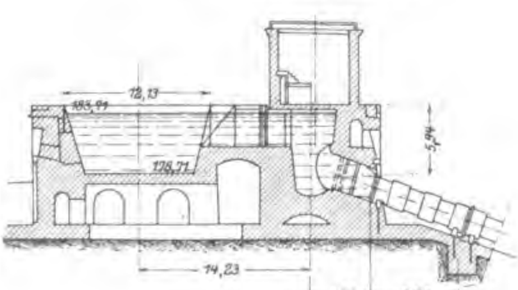


Fig. 349. Pressure Chamber. Vizzola Plant.

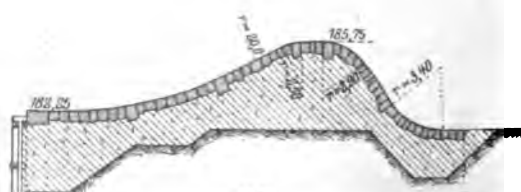


Fig. 348. Cross-Section of Dam. Vizzola Plant.

generator sets. The turbines are rated at 1900 HP. These are of the horizontal shaft type with twin runners. They are controlled by oil governors, and use is also made of fly wheels to overcome momentary load fluctuations, their inertia being 80,000 ft. lbs.

Generators. Each turbine drives a 1000 kw., 2300 volt 25 cycle, three phase generator running at 375 r. p. m. Two exciter-sets are also provided, and consist each of a 200 kw., 120 volt direct current generator, driven by separate horizontal shaft, single-runner, spiral case turbines, of a rated capacity of 375 HP at 500 r. p. m. The output of each exciter is sufficient for the entire plant. The switchboard is placed on a gallery overlooking the generator room and the switches are on the second floor. The transformers are of 1110 kw. capacity, oil insulated, water-cooled type, by means of which the current in stepped up to a tension of 66000 volts.

Transmission line. There is one tower line, 62 miles in length carrying two circuits. The conductors are six strand hemp center copper cables, equivalent to No. 0 B. & S. gauge. They are hung on suspension type insulators from the cross-arms of steel towers 60 ft. in height as illustrated in fig. 345.

194. Stave Falls Plant. Canada.

(Head 90 ft.)

This plant has recently been put in service by the Western Canada Power company. The Stave River from which water is derived flows out of Stave Lake which will eventually provide an available storage of 14,000,000,000 cu. ft.

Dam. This structure which in reality is a sluice dam, is 160 ft. long and 62 ft. high, providing five sluice ways 22 ft. in width each. The intake dam which is of concrete has a length at the crest of 145 ft. a height of 75 ft. and a top width of 40 ft., fig. 346.

Pipe lines. The two main pipe lines are 165 ft. long and 14 ft. 6 in. in diameter. The two penstocks for exciters are 4 ft. in diameter.

Power house. The Stave Falls Plant power house is 110 ft. long and 104 ft. wide, and accomodates two 13,000 HP turbines direct-connected to 7500 kw. generators. There are also two 500 HP turbines direct connected to the exciter units. See Fig. 346.

There are furthermore six 60,000 volt 3000 kw. step up transformers which will deliver current over a double transmission line. The ultimate capacity of this plant will be 50,000 HP.

195. Vizzola Development. Italy.

(Head 92 ft.)

This plant of 20,000 HP capacity is one of the most important developments in Italy. Its layout is shown in fig. 347.

The necessary water power is furnished by the Tessin River, between Somma and Vizzola. The distance between these two localities is 18.6 miles, with a fall of 98 feet.

Dam and Canal. The dam has a height of 42 feet and is 214 ft. long, fig. 348, and is provided with gates of 6700 cu. ft. per second discharge

capacity. The gates are hand operated and 30 in number. The canal has a slope of 6 feet per mile, and the average velocity of the water is 4.6 ft. per sec. The water is carried to the pressure chamber and before arriving at the latter, passes over a concrete-canal bridge. Fig. 351.

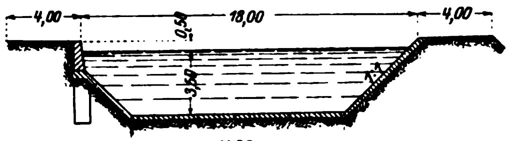


Fig. 350. Cross-Section through Canal. Vizzola Plant.

Pressure pipes. From the pressure chamber, Fig. 349, the water is carried to the turbines by means of 12 pressure pipes, all protected by screens. The gates are hand operated.

Power house. The power house is 306 ft. in length and 61 ft. in width. It contains 10 turbo-generator sets, one of which serves as a reserve unit.

There are also two exciter sets. Eight of the turbines were built by Riva, Monneret and Co., two by the firm of J. Voith. They are of the horizontal shaft type and run at 187 r. p. m. Each turbine has a capacity of 2000 HP, and drives a revolving field alternator, generating three phase current at 11,000 volts 50 cycles. The exciter furnishes 1300 amperes at 110 volts.

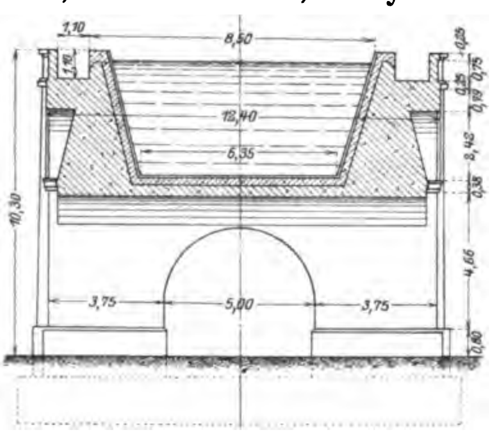


Fig. 351. Cross-Section of Canal-Bridge. Vizzola Plant.

Transmission lines. The transmission lines are carried over steel towers. Transformer stations reduce the voltage to 3600 volts. The distributing system has an aggregate length of 162 miles.

The current is sold for 77.2 dollars a kw. year to consumers using less than 5 kw., and 30.80 dollars for consumers using over 700 kw. for each day of 12 hours.

This plant runs in parallel with the plant at Turbigo, of 3000 HP capacity, and the Castellanza plant, 5500 HP, which acts as a steam reserve.

196. Central Georgia Power Co's Hydroelectric Plant.

(Head 100 ft.)

This plant is located midway between Atlanta and Macon, in Georgia. The necessary power is derived from the Ocmulgee River, which like other streams of the same region, is shallow and has a fall of about 4 ft. to the mile.

Dam. The necessary operating head is obtained by means of a concrete dam and adjacent earth embankments, having a total length of 1720 ft. The structure is located in a narrow part of the stream, the width from shore to shore at that section being about 300 ft., the height above foundations varying from 35 to 127 ft. One part of it, 420 ft. long, has the form of a spillway, the crest of which is 3 feet below the crest of the dam

proper. Provision is made for flashboard along the entire crest of the spillway. Two permanent hand operated sluice gates are located next to the power house.

Penstocks. These are built of solid concrete, as shown in the cross-section of the power house, Fig. 352; they are 12 ft. square, with the corners cut off; they terminate in a wheel chamber formed by the casing of the turbine.

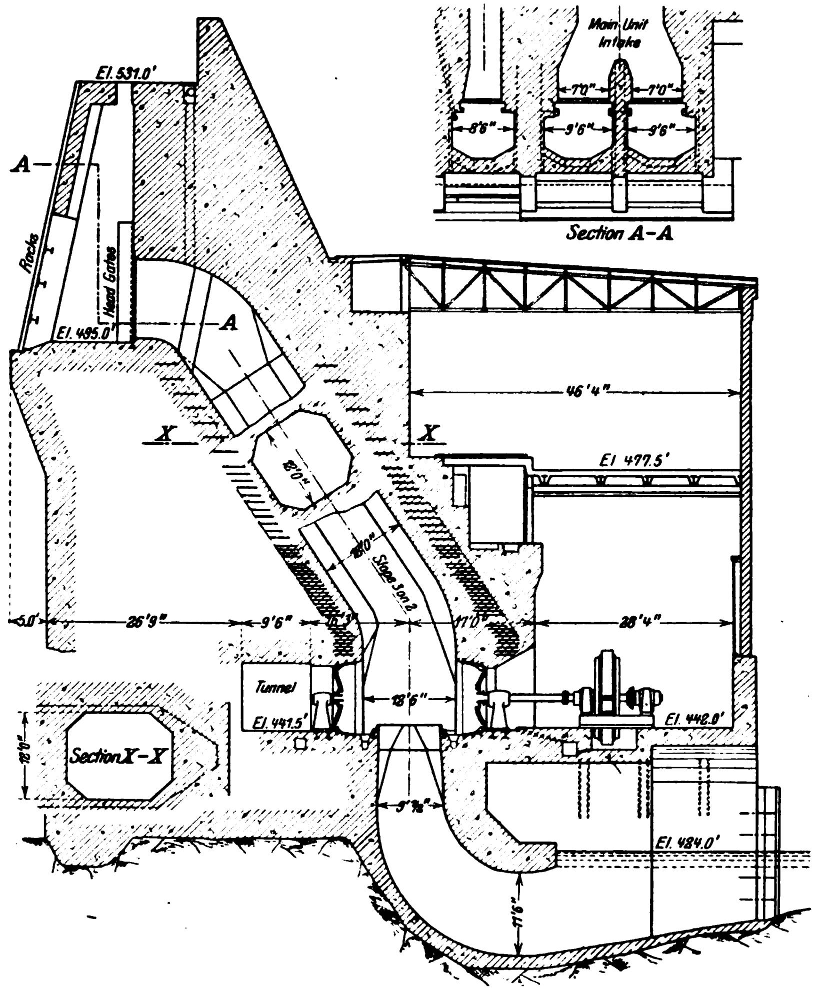


Fig. 352. Vertical Section on Center Line of one of the Main Units. Central Georgia Power Co.

Power house. The power house is 194.5 ft. long and is composed of two floors, the width of the lower being 28′-4″, and the width of the upper floor 46′-4″. The frame is a steel structure, with brick curtain wells. Room is provided for six water wheel units, of the McCormick horizontal type, each consisting of a pair of runners 39 in. in diameter, equipped with Smith wicket gates. Their capacity is rated at 5500 HP at a speed of 300 r. p. m. under a head of 100 ft. The units are spaced 25 ft. center to center. Each unit is connected to a 3000 kw., 2300 volt, 3-phase 60 cycle generator. The exciters, of which there are two, have a capacity of 250 kw. and gene-

rate current at 250 volts. These machines are directly connected to their respective water wheels, 24 inches in diameter, rated each at 550 HP with a speed of 550 r. p. m. The transformers and high tension switching apparatus are located on the upper floor, whereas the main switchboard is placed on a gallery overlooking the generator room, fig. 353.

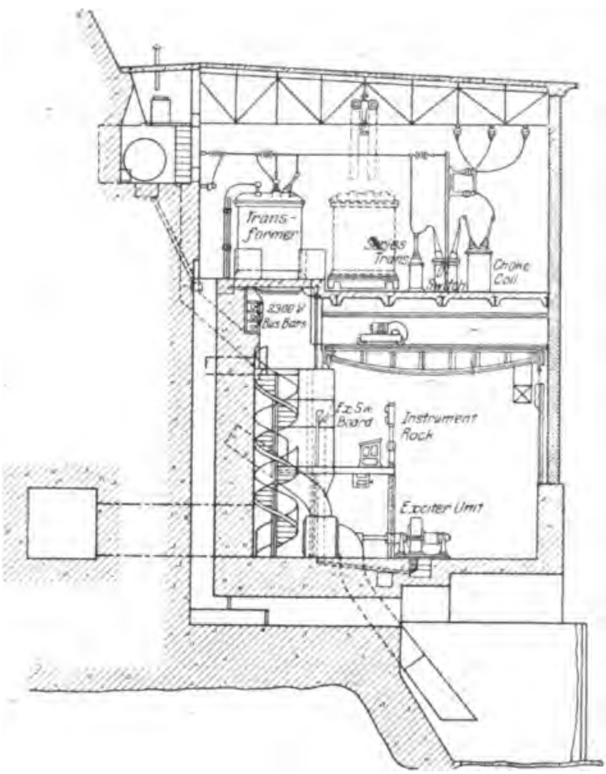


Fig. 353. Transverse Section of Power House, showing Exciters and Controlling Apparatus. Central Georgia Power Co.

Transmission line. The current generated is distributed over a radius of 50 miles.

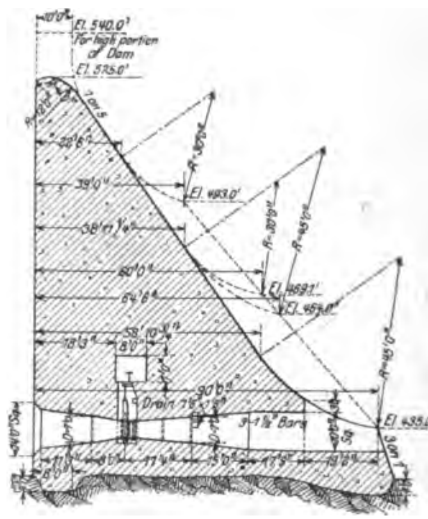


Fig. 354. Cross Section of Dam at Sluice. Central Georgia Power Co.

covering thereby the territory in which are located numerous cotton mills and other industries. For this purpose, the current is transmitted at a tension of 66,000 volts over a line supported by steel towers.

197. Wolf Creek Plant, United Missouri River Power Co.

(Head 114 ft.)

This plant, which is also known under the cognomen of Capital City plant, is located about 30 miles north of Helena. The Missouri River at this point passes through a short and narrow canyon, the width of which, along the foundations of the dam built in this channel is 700 ft., whereas the total top length of the dam is almost 1400 ft.

Dam. This structure has an average height of 130 feet from the foundations to the bulkhead section, and is capable of sustaining a head of 114 ft. It is of concrete, with imbedded rock in the more massive sections. The most notable feature of this dam is that it is provided with an inspection tunnel and a complete drainage system (see fig. 355). The dam is also provided

with a bridge and flashboard system, consisting of removable planks manipulated either from the bridge or from a floating barge.

Penstocks. There are 7 main penstocks, and 3 exciter penstocks each feeding one unit. These penstocks are imbedded in the concrete masonry, their upper openings being controlled by electrically operated steel gates. As each penstock is independent, any one of them can be stopped with a stop log, thus closed for inspection or repairs.

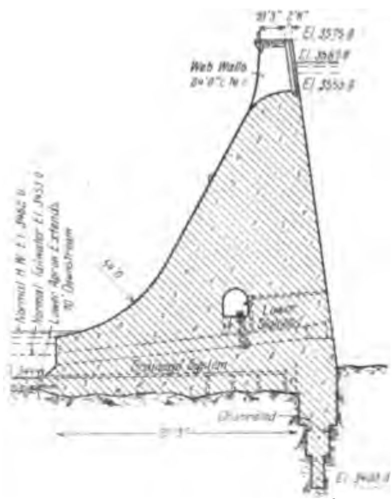


Fig. 355. Cross-Section of Dam.

Power house. The edifice is on the west bank of the river, and built parallel with the line of the dam, while the transformer house is perpendicular to the direction of the dam; the power house contains 7 main and 3 exciter units. Each hydraulic machine is composed of 2 runners, mounted on a single shaft, and each is of 8800 HP capacity at full gate opening. Each wheel is provided with its individual scroll case and both wheels discharge into a common draft tube. The governors are of the Lombard vertical type.

Each turbine is direct connected with a 5000 kw. three phase machine generating current at 6600 volts. There are besides 3 exciter units and one auxiliary unit, each direct connected with spiral case water wheels.

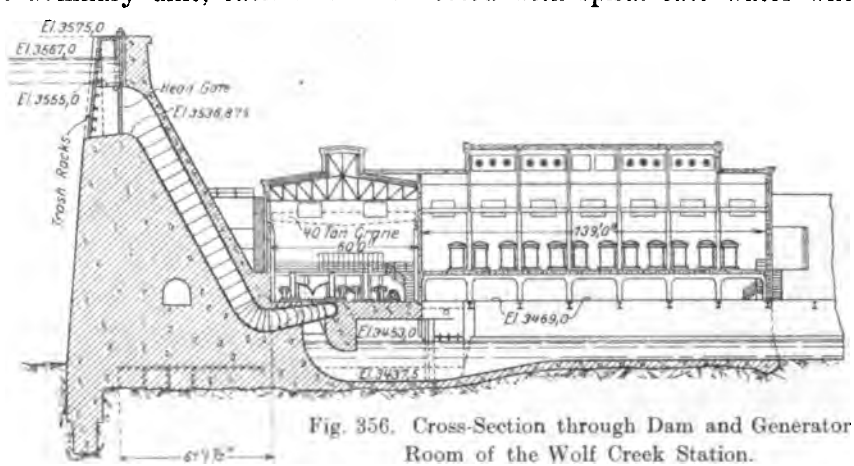


Fig. 356. Cross-Section through Dam and Generator Room of the Wolf Creek Station.

The transformers and other high tension apparatus are located in the transformer house. A cross-section through the power house is shown in fig. 356.

Transmission line. The electrical energy generated at the Wolf Creek Station is transmitted at 70,000 volts, and combined with the energy generated at the Hauser Lake and Canyon Ferry Plants, owned by the same Company. The line is composed of two circuits, and is located on private right of way.

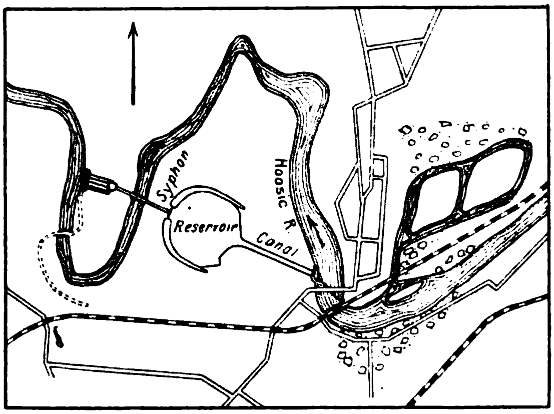


Fig. 357. General layout of the Schaghticoke Development.

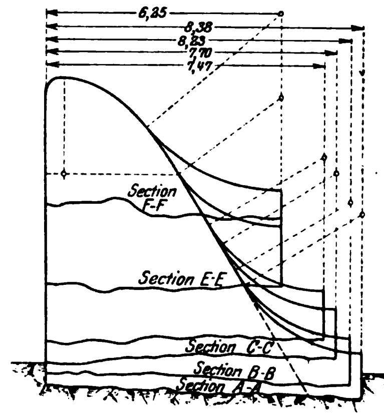


Fig. 358. Cross-Section of Dam. Schaghticoke Development.

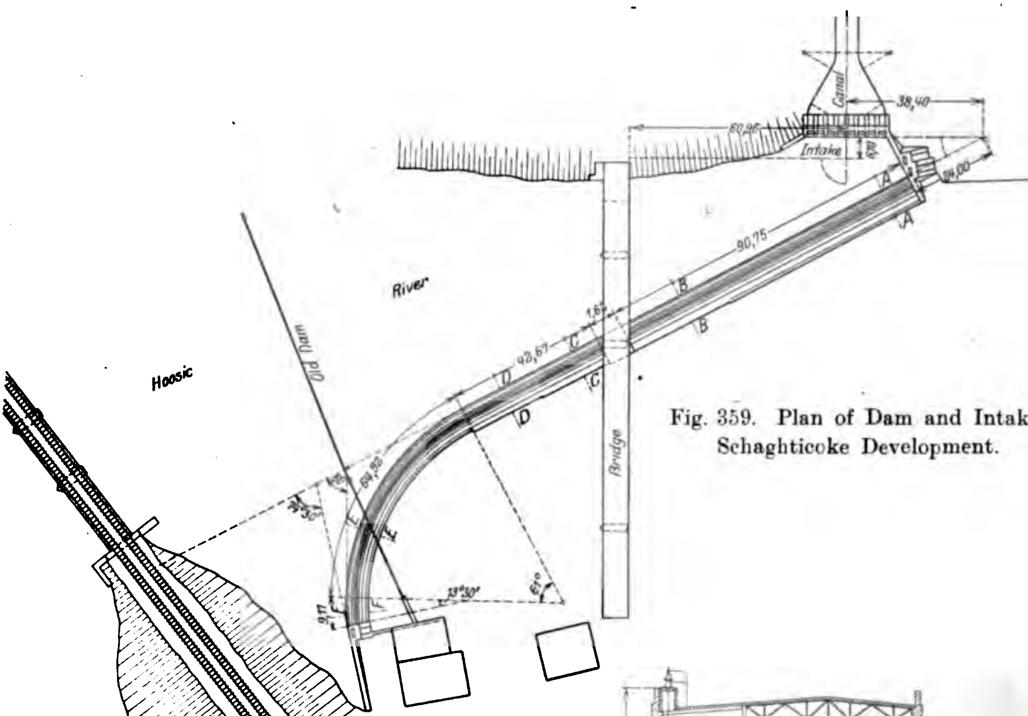


Fig. 359. Plan of Dam and Intake. Schaghticoke Development.

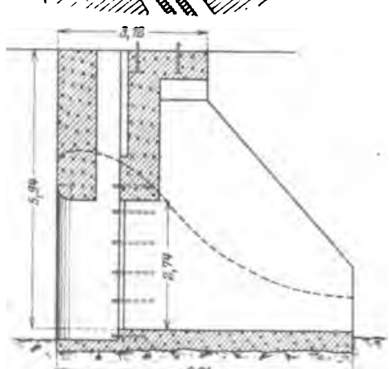


Fig. 360. Cross-Section through Canal Intake. Schaghticoke Development.

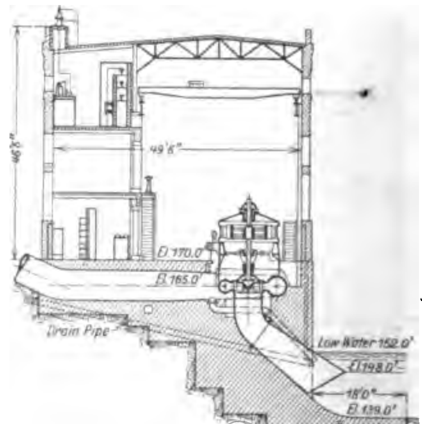


Fig. 361. Cross-Section through Schaghticoke Power House.

198. Hoosic River Development, Schaghticoke. New York.

(Head 150 ft.)

This plant is interesting not only from the technical, but also of the power market point of view: within a radius of 21 miles is situated Schenectady, the home of the General Electric Co.

The general layout of the development is illustrated in fig. 357.

The Hoosic River, which furnishes the necessary power, rises in the Berkshire Hills, Mass., and discharges into the Hudson River about 14 miles from Troy, and 90 miles from its source. Its drainage area above Schaghticoke is about 600 sq. miles. The discharge of the river varies from 1 00 cu. ft. per sec. in the dry season to 17,000 cu. ft. per sec. (Oct 1907).

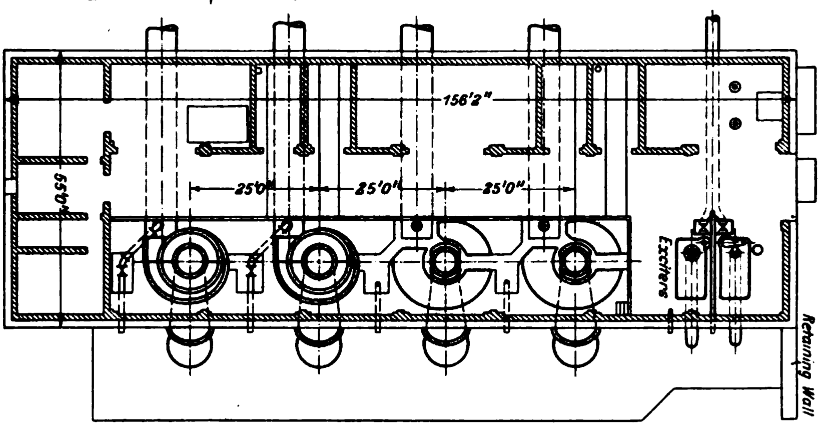


Fig. 362. Plan and Section of the Schaghticoke Power House.

Dam and intake. The dam is of the ogee type, fig. 358, about 680 ft. long, its height varying from 21 to 24 ft. above the ground. It is made of concrete mixed in the following proportions: 1 part cement, 3 parts sand, and 5 parts of broken stone. The lake created by this dam extends over 145 acres and its storage capacity is about 1230 acre feet; this is fed by another reservoir at Johnsonville, five miles up stream, the latter's capacity being 8640 acre ft. A system of flash boards has been adopted in order to raise the crest of the dam by 3 feet. The canal intake consists of eight 6 $\times$ 9 ft. gates. The canal itself measures about a half a mile in length and carries the water to a second concrete structure, protected by racks and an ice chute. The penstock which starts from this points, has the form of an inverted siphon, about 850 ft. long, 12.5 ft. in diameter, built of steel riveted plates $\frac{3}{8}$ in. thick. This penstock carries the water to a cylindrical surge tank, also of steel, 40 ft. in diameter and 56 ft. high.

Pressure pipes. From this tank, 5 steel pipe feeders extend down to the power house, 4 for the main units and 1 for the exciter set, the diameter of the main pipes being 6 feet and that of the exciter pipe 2 ft. All these pipe lines are provided with expansion joints.

Power house. This measures 55 ft. in width and 156 ft. in length. The sub-structure is of concrete, the superstructure has brick walls reinforced by a steel frame. It contains 4 turbo generator sets. The turbine are of the Francis vertical shaft type, rated at 5000 HP under a head of

146 ft. and a speed of 300 r. p. m. They are directly connected to their respective 3000 kw. alternators generating 3-phase current at 4400 volts, 40-cycles and 300 r. p. m. The exciter units consist of two 150 kw dynamos, type M. P. C. General Electric Co., generating 250 volt direct current at 600 r. p. m. These machines are driven by 250 HP horizontal shaft Pelton Francis turbines and furnish 600 amperes at full load.

The switchboard composed of 11 panels is located on a gallery above the generator room. About 150 kw. at 4400 volts are used at Schaghticoke for local consumption. The rest, or a little less than 12,000 kw. is transmitted to Schenectady. There are 4 transformers of 3000 kw. capacity, 4400/32,000 volts.

Fig. 363. Schaghticoke Transposition Tower.

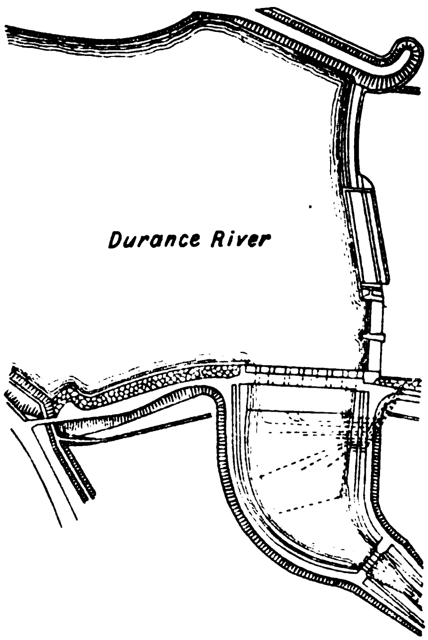


Fig. 364. Dam, Intake and Sedimentation Basin. Durance River Development.

Transmission line. There are two circuits of 000 conductors, and the current is transmitted to a sub-station located at the G. E. works. These two 3-phase lines are supported by steel towers. Between the plant at Schaghticoke and the sub-station at Schenectady there are 197 towers, of which twelve are of partial transposition, and 6 of complete transposition. The main towers are 37 ft. high. Three special towers about 96 ft. high are used near Schenectady, whereas for the crossing of the Hudson River, towers nearly 76 ft. high have been used.

199. The Durance River Plant. France.

(Head 164 ft.)

This plant, located at Ventavon, in the extreme eastern part of France, was built with a view of feeding the electrical system supplying the region between Avignon and Marseilles.

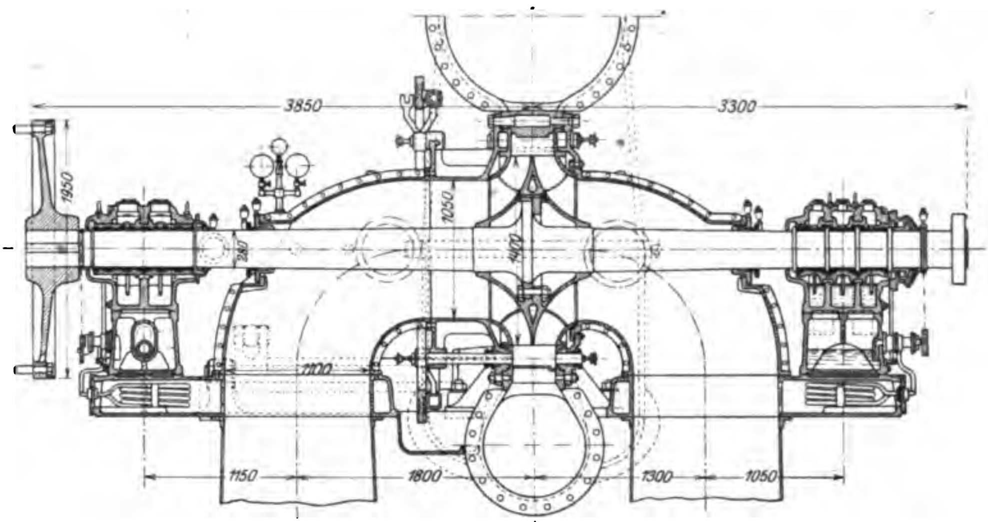


Fig. 365. Vertical Section through Turbine Durance River Development.

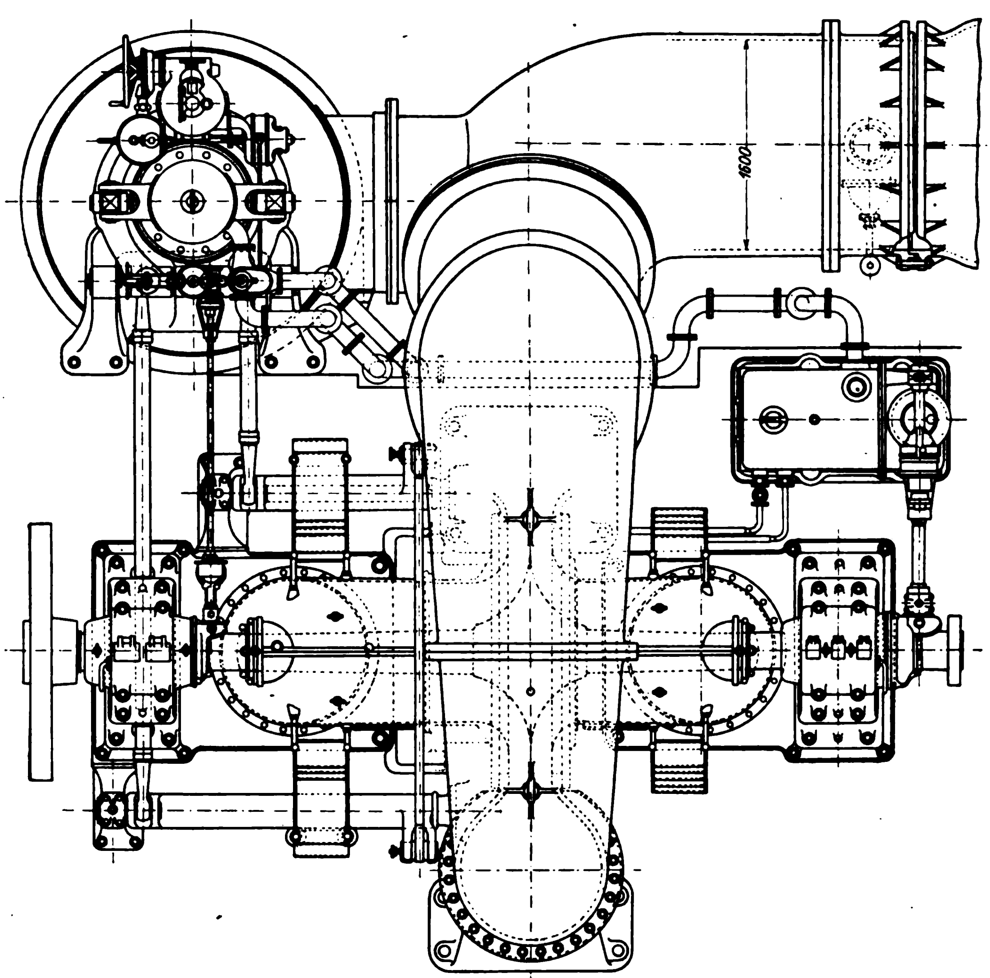


Fig. 366. Plan of Turbine Durance River Development.

The water is diverted from the Durance river, a stream noted for the excessive amount of detritus and mud which it carries. Therefore the intake fig. 365; has been designed rather as a mud catch basin.

Dam. This is a curved masonry structure made of three parts as shown in fig. 364: gateways, spillway and dam proper. On the side nearest the forebay, three sluice ways are provided in which metal gates are operated. The smallest gate, placed against the spillway, has for object the passing of ice and floating debris. The spillway is 175 ft. long. The canal intake is composed of 8 gateways, the gates of which are each composed of three leaves intended to be operated independently.

The forebay has a concrete lined floor and a number of pipes end to it, opening upward and flush with the floor. These pipes are connected to a feed main, which takes water from the river above the intake and passes it through these openings to sluice away the sediment.

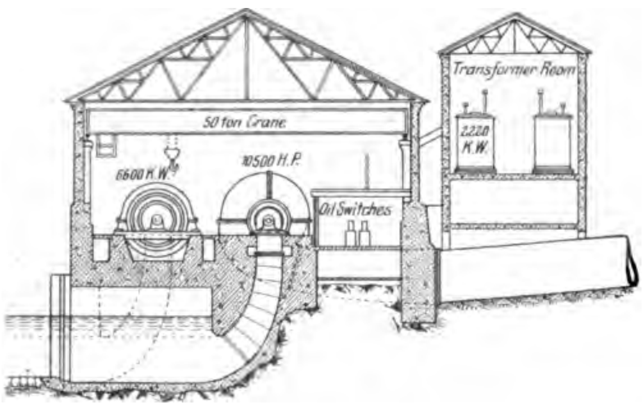


Fig. 367. Shawinigan Falls Power Plant.

The canal, which is trapezoidal in form with an area of about 322 sq. ft. is 8.7 miles long with a fall of 1.28 ft. to the mile, and carries 1975 cu. ft. per sec.

Penstocks. The forebay proper is located 1320 ft. from the power house. Four 14.8 ft. gates admit the water to the penstocks 7.51 ft. in diameter. Two additional penstocks 3.3 ft. in diameter supply the water to the exciter sets.

Power house. The building is 285 ft. long and 67 ft. wide. It accomodates our main turbines of 6200 HP each and two 300 HP exciter turbines. The main turbines illustrated in fig. 364 and 365, are of the Francis type with horizontal twin runners 4.6 ft. in diameter and a normal speed of 300 r. p. m. The casings are of spiral form, built up of four pieces, each casing supplying a pair of runners which discharge into separate draft tubes.

Four means of regulation are provided: automatic oil control, hand operated hydraulic pump, hand operated screw and electrically driven screw.

The alternators are 4500 kw. machines, generating at 7500 volts and are direct-connected to the turbines by means of a semi-flexible coupling.

The transformer house is connected to the power house by an underground conduit through which the feeders pass.

The current is stepped up to 55,000 volts by means of 12 water-cooled oil transformers.

The transmission line is 37 miles long and connects this plant with the one at Brillane.

200. Shawinigan Falls Power Plant. Canada.

(Head 170 ft.)

This power plant is situated on the St. Maurice River, about 90 miles from Quebec. The St. Maurice River, which has a total length of over 400 miles, has a drainage area of 18,000 square miles and is supplied from several lakes. The flow varies from 20,000 to 26,000 cu. ft. per sec.

Intakes. The intake canal is 100 ft. wide, 20 ft. deep and 1000 ft. long. The end of the canal is closed by a concrete wall from which the water is carried to the wheels by means of steel penstocks 9 ft. in diameter. The concrete wall is 40 ft. high, 12 ft. wide at the top and 30 ft. wide at the bottom. The head gates which form part of it are operated by hydraulic cylinders.

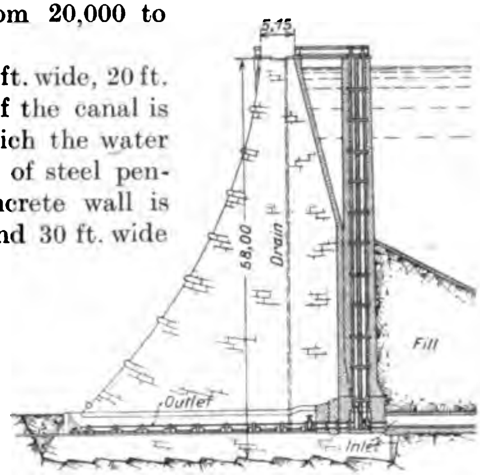


Fig. 368. Section through Dam and Valve Chamber, Urfttalsperre Plant, Germany.

Power house. The power house, fig. 367, contains 3 double units of 6000 HP, of the horizontal type. They are direct connected to 5000 HP generators of the rotating field type, giving two-phase 30-cycles current at 2200 volts, and 180 r. p. m.

Two water wheels of 10,500 HP each, and driving 6600 kw. generators have been installed later, bringing up the total capacity of the plant to 38,000 HP.

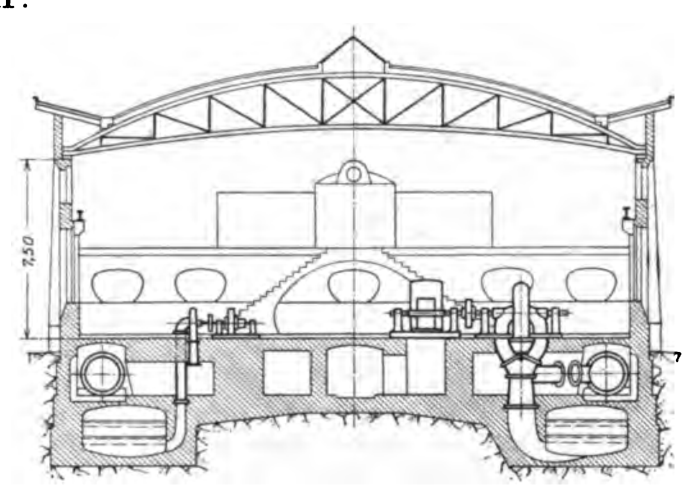


Fig. 369. Transverse Section through Power House, Urfttalsperre Plant.

The exciters are driven by two separate 400 HP turbines to which the water is carried by a separate penstock. The power is transmitted by means of a 25,000 volt and a 50,000 volt transmission line.

201. Urfttalsperre Hydroelectric Plant. Germany.

(Head 230 to 360 ft.)

This development is located at Heimbach on the Rur and draws its energy from the river Urft which has a drainage area of 145 sq. miles above the dam.

Dam. The dam, fig. 368, is made of cyclopean masonry, 190 ft. high, the width at the bottom being 165 ft. and at the top 17 ft. The structure is arched in plan with a total length of about 1000 ft., 300 ft. of which are used for spillway. Two discharge pipes are provided at the bottom of the dam for the purpose of drainage.

Tunnel. A tunnel of 8850 ft. length and 60 sq. ft. cross-section area is cut through the mountains, and conveys the water to a collecting basin, which at the same time serves the purpose of a stand pipe. From here, two pressure pipes carry the water to the power house.

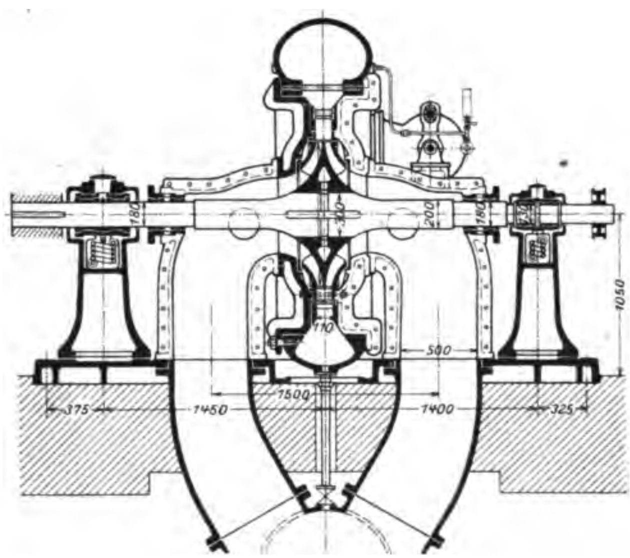


Fig. 370. Cross Section of Turbine, Urfttalsperre Plant, Heimbach, Germany.

Power house. The Urfttalsperre power house is a masonry structure 95 ft. long and 75 ft. wide. There is a separate switching room 75 ft. by 30 ft. The turbo-generator units are placed in two parallel rows as indicated in fig. 369. The generator room accomodates six units consisting of double flow horizontal Francis turbines with a capacity of 1550 PH at 230 ft. head and 2000 HP at 360 ft. head. The speed is 500 r. p. m.

They are direct connected to 1370 kw., 5000 volt, 50 cycle, 3-phase generators. There are also two exciter units of 200 HP each running at 900 r. p. m. The exciters have a capacity of 135 kw. and deliver direct current at 225 volts. The voltage is raised to 34,000 volts by means of the transformers.

Transmission line. There are four lines: 38.5, 24.0, 16 and 21 miles respectively in length. The poles are built of structural steel. The complete hydro-electric transmission system, including the transmission lines cost approximately $ 2,500,000 (dollars).

202. Hydroelectric Plant at Strangfjord. Norway.

(Head 290 ft.)

This plant is located about 80 miles north of Bergen, at the head of the Strangfjord, the water storage being obtained in a chain of six lakes.

Dam. A masonry dam, 321 ft. in length, has been constructed at the outlet of the lower lake. Sluice gates raised by hand regulate the quantity of water delivered to the turbines.

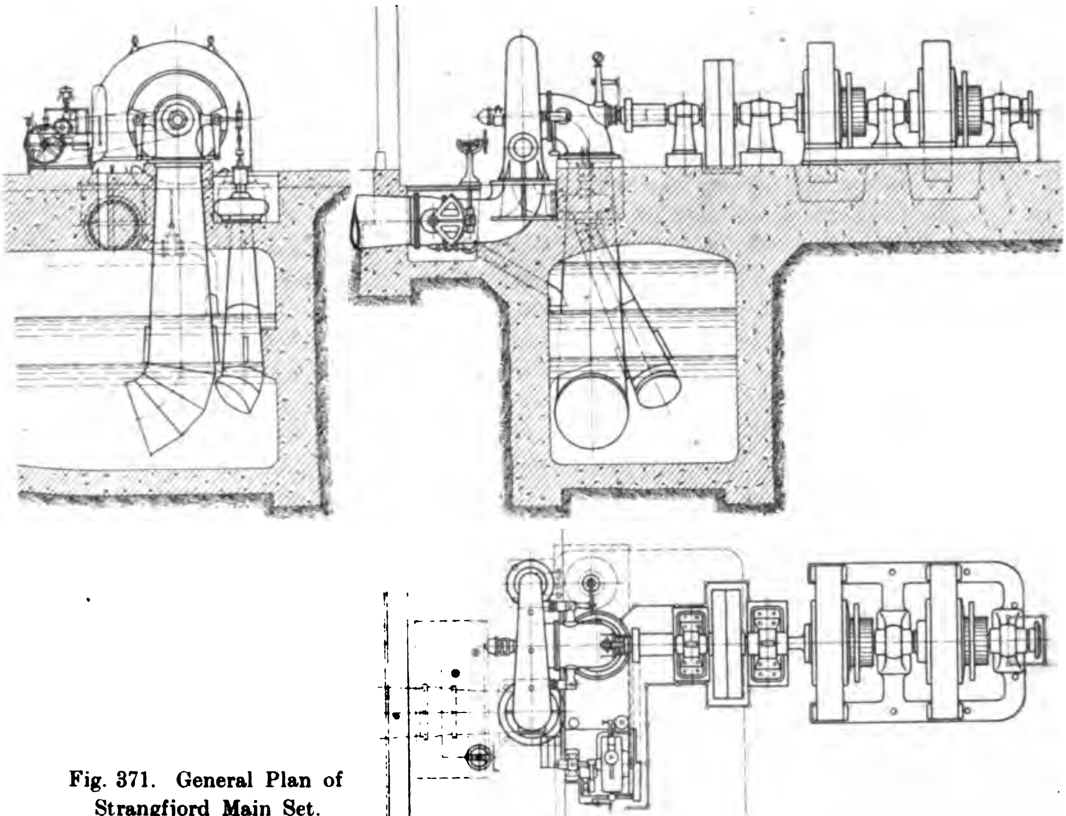


Fig. 371. General Plan of Strangfjord Main Set.

The water is carried about 1640 ft. through a wooden conduit having a cross section of 7.5×5 feet and a slope of $\frac{1}{2000}$, then through a tunnel 656 ft. in length to the penstock, whence it is carried down to the power house by means of 4 riveted steel pipes, 36 inches in diameter and 1230 ft. long.

Power house. The power house, fig. 371, accomodates one Francis water turbine rated at 3000 HP, directly connected to a pair of Dick Kerr direct current generators operated in parallel, and generating 8000 amperes at 275 volts. The speed is 300 r. p. m. The regulation of the turbine is effected by an oil pressure governor connected by a by-pass, which opens when the turbine is shut down suddenly. There are 28 movable guide vanes on the turbine. They are opened and closed by means of a steel ring which carries a number of sliding blocks of bronze into which the crank pins of the guide-blades are fitted. This ring is worked by the governor through a pair of levers on opposite sides of the turbine.

203. Plant of the Mohawk Hydroelectric Co. Ephratah, N.Y.

(Head 300 ft.)

The plant of the Mohawk Hydro-Electric Company is located at Ephratak, New York, and the station receives its water supply from Garoga Creek. On the headwaters of the creek are Peck and Garoga Lakes, which furnish storage for the development.

Dam. One has been built at the outlet of Peck Lake; the length of the crest is 1700 ft. and its maximum height 40 ft. At the north end of the lake, there is a timber crib and earth fill dike, with the same height as the one above, but the whole length of which is 1900 ft. Both of these dams were built to impound the water in the lake, forming a storage capacity of some 1,250,000,000 cu. ft. of water.

The forebay dam, situated 10 miles down stream has a maximum height of 60 ft., the length at the crest being about 750 ft.

Tunnel and penstocks. From the forebay, the water is conveyed through a concrete tunnel 400 ft. long and 7 ft. diameter, thence through a continuous banded wood stave pipe 78 inches in diameter, thence through a 96 inch pipe of same material for 1460 ft. and finally through a steel penstock of 96 inches in diameter. A reinforced concrete surge tank 25 ft. in diameter 50 ft. high has been provided and located about 2300 ft. from the power station.

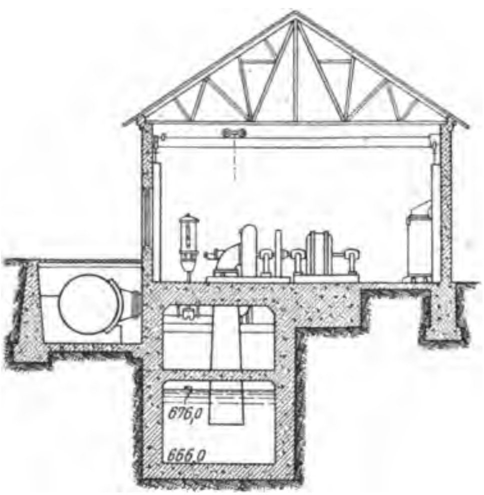


Fig. 372. Cross-Section through Mohawk Station.

Power house. It is a masonry structure 103 ft. long and 38 ft. wide, fig. 371, and contains three 1750 HP. Francis type turbines running at720 r. p. m. and regulated by Lombard governors. They are direct connected to 1250 kw. 2300 volt 60 cycle 3-phase alternators and are provided also with 5 ton fly wheels. The exciter unit of 50 kw. is driven by a Pelton wheel.

The 7 transformers are of the water-cooled, oil-insulated type, and the ratio of transformation is 2300/22.000 volts.

Transmission lines. Power is transmitted to Johnstown and Gloversville, distance of 10 miles. There are four wires besides the ground wire and the telephone circuit, the fourth wire being held in reserve.

The towers are spaced about 450 ft. apart, these being of the *A* frame, four logged type.

204. The 500,000 H. P. Queenston-Chippawa Project in Ontario.[1]

(Heat, 305 feet.)

An ultimate development of 500,000 hp., produced by nine 45,000-kva. generators, is contemplated for the Queenston-Chippawa project, wich has

[1] E. T. J. Brandon — Electrical World. Vol. 77. No. 13.

been under construction for about three years by the Hydroelectric Power Commission of Ontario. This project, which is the biggest under way at the present time, when completed will utilize nearly the full drop between Lakes Erie and Ontario near Niagara Falls. Construction is now under way on half of the ultimate plant. The generators and the turbines, of which five have already been ordered, will be larger than any in use at the present time. The 50,000-hp. turbines will operate at a head of 305 ft. (93 m.).

The five contracts which have been placed are for 45,000 kva., 12,000 volt, three-phase, 25-cycle, $187^1/_2$ r.p.m. vertical generators. Three of the machines are being manufactured by the Canadian Westinghouse Company and two by the Canadian General Electric Company. One of these is now ready for shipment. Direct-connected exciters and thrust bearings are mounted directly above the generators. The air system is completely inclosed so that the cooling medium can be taken from and discharged to the outside air without mixing with the air inside the station. It is an interesting fact that the weight of air passing through one of these generators every three hours will equal the total weight of the generator itself. At full load, about 100,000 cu. ft. (2,800 cu. m.) of air per minute will be required. In the present installation there will be fifteen 15,000 kva., 25-cycle, single-phase transformers. These are connected in delta on the 12,000 volt side and in star on the high-tension (63,500 volt) side. Taps on this side allow the voltage to be changed from 110,000 to 132,000.

Provision is made for operating on the unit system. That is, one generator, one transformer and one transmission line will be considered as a unit and each will be able to carry the same load. In case of emergency a unit may be operated at full load of one generator without utilizing either the high-tension or the low-tension buses. The units may also be operated in parallel on either the high-voltage or the low-voltage bus. For this purpose current-limiting reactors are placed between generators on the 12,000 volt bus. These reactors may be shunted out by means of oil switches in case it is desired to feed directly from one generator to the transformer of another generator. There is an emergency bus on the high side and provision has been made for an emergency bus on the low side if it becomes desirable in the future. Any one of the generators may be connected to the main bus or straight through to its transformer bank. Likewise the transformer bank may be connected to the main bus or straight through to the generator. The same arrangement holds on the high-tension side of the transformer bank, it being possible to tie the outgoing lines straight into the bank, to the main bus or to an emergency bus. Similarly the high-tension side of the transformers can be connected straight through to the line, the main bus or the emergency bus (see fig. 374).

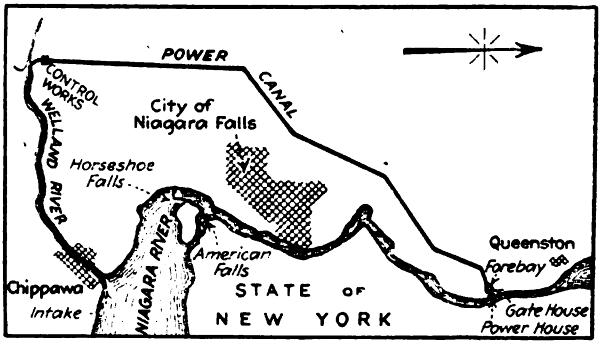


Fig. 373.

All electrical equipment contracted for to date, except some minor accessories, is estimated to cost about \$ 4,000,000 and will be made in Canada.

Generating Station. The entire equipment belonging to one unit, consisting of a generator, high-voltage and low-voltage switching apperatus and transformers, occupies a 50 ft. (15 m.) length of the station. The five units at present on order will require 250 ft. (76 m.) of the station and 100 ft. (30 m.) more will be used for erection space and surplus equipment, making the building for the present 350 ft. (106 m.) long. The ultimate length will be about 650 ft. (200 m.).

The main generator room will be about 60 ft. wide by 60 ft. high (18 m. × 18 m.). An interesting feature of the construction is that the floor of this room is level with the top of the generator frames. The space between this floor at the bottom of the generator is used for cooling-air ducts, power leads and low-voltage and high-voltage switching apparatus is approximately 90 ft. (27 m. × 30 m.) high.

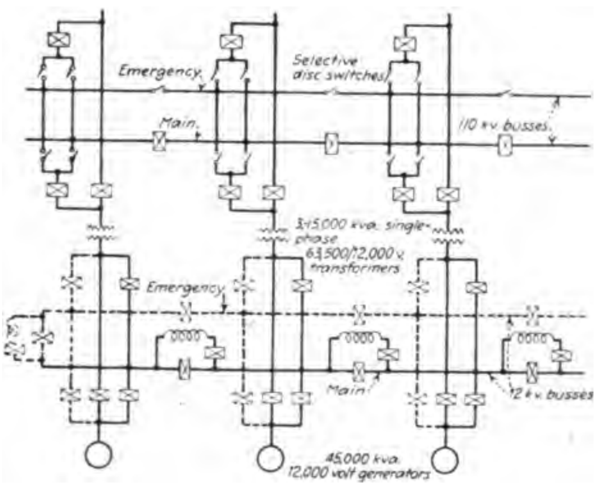


Fig. 374.

The generator leads run beneath the generator-room floor to oil switches in an adjacent room on the same level. The low-voltage gallery for main and emergency buses is just above this room and the gallery for the low-voltage transformer switches is just above the bus gallery. Leads from this switch gallery pass into the transformer room, the floor of which is level with the generator floor. All high-tension oil switches are on a floor above the transformer room. On the top floor just this are the main 110-kv. bus, lightning arresters and line disconnecting switches. The emergency high-tension bus is in a room below the oil switches and furthest from the generator room. The control room is above the generator room and overlooks the river.

Hydraulic Installation. Interest attaches to the hydraulic part of the installation on account of certain details. Among the important features which diverge from standard practice are the large single-leaf control gates near the beginning of the canal, the 50,000 hp. turbines—which are the largest ever built—the location of the power house, the record size of the Larner-Johnson valves and the special details of ice skimmers and track racks. This plant takes water from the Niagara River about $1^1/_2$ miles (2.4 km.) above the falls and delivers it, through a canalized river and a dug canal, a total distance of $13^1/_2$ miles (21.7 km.), to the power house, which is under a bluff on the lower Niagara River 5 miles (8 km.) below the falls. The upper $4^1/_2$ miles (7.2 km.) of the canal is in the Welland River, the flow of which is reversed for that distance, the canal section, cut partly through rock and

partly through earth, being $8\frac{1}{2}$ miles (13.6 km.) long. A single motor-operated vertical-lift roller gate for the purpose of controlling or entirely shutting off the flow will be installed at the upper end of the canal near Montrose, where the earth section of the canal merges into the rock section. The combination of span and head make this gate the largest ever built, and particular attention has been paid to the roller design in order to insure bearing loads well within safe limits. The clear width is 48 ft. (14.4 m.), and the height of the gate is $42\frac{1}{2}$ ft. (12.7 m.). It has been decided to omit permanent gates in the screen house and the penstocks as it is thought that Johnson valves alone will be sufficient. To take care of any failure in the valves, removable gates made up in sections will be provided, which can be lowered into any penstock entrance by an electric traveling crane.

An elaborate ice skimmer has been made and will be installed later if necessary. This consists of a reinforced-concrete horizontally pivoted leaf which can be raised or lowered to skim floating ice of the surface into a discharge channel, at a same time allowing clear water to pass underneath to leaf.

Turbines. The turbines are of the vertical, spiral-case, single-runner type and will operate at a speed of $187\frac{1}{2}$ r.p.m., which means a specific speed of 36. The maximum guaranteed efficiency is 90 per cent, although in view of recent practice it is expected that this efficiency will be exceeded. The inlet diameter of the scroll case is 20 ft. (6 m.), and the diameter of the runner is 10 ft. 5 in. (3.1 m.) at the inlet. An open space has been left in the power-house foundation below the runner in order that it may be taken out from below by removing a section of the draft tube, thus obviating the necessity of dismantling the generator when a renewal of the runner is to be made. The runners and spiral casings are cast steel, and a test pressure of 260 lb. per square inch (18 kg. per sq.cm) is required on the latter.

The centrifugal-head relay valve controlled by each governor will be installed on the generator floor, while the main automatic valve control will by placed directly under the governor stand at the level of the turbine-regulating cylinder. This arrangement has the advantage of requiring short piping between the main valve and the regulating cylinders and gives freer access for repairs and maintenance. The pressure fluid will be water, probably treated with bichromate of potassium to prevent rusting and at the same time give a lubricating quality to the water.

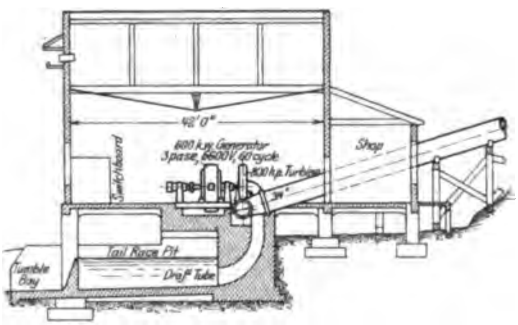
Adjacent to each generator a control pedestal will be set up, and on this the various indicating instruments will be mounted. The control room will be so connected with the local control and indicators that the operator on the generating floor can handle the machine while in touch with the chief operator. Telegraph communication similar to a ship telegraph and a loud-talking telephone communicating with the control room are the principal means of transmitting signals. In addition to this a lamp mounted at the top of a column over the control pedestal will enable the chief operator to call the floor operator to any unit as required. The air-brake and Johnson-valve control will also be mounted on the pedestal. The location of this pedestal adjacent to the governor places the control of all the important operations of the unit within easy reach of the operator, while he is at the same time in communication with the control room.

205. Ketchikan Plant. Alaska.

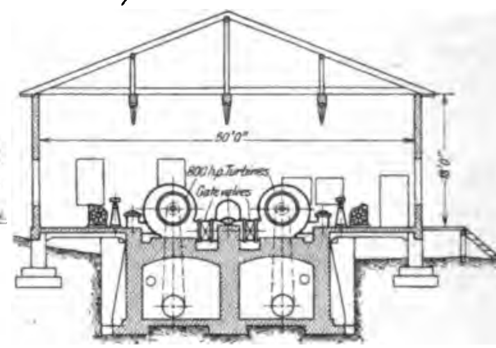
(Head 340 ft.)

The source of power is a lake about one mile long by half a mile wide.

Dam. This is of the "K-truss" form and closes up the valley. It is made of logs, 30 ft. long at the bottom and 65 ft. at the top, being 25 ft. in height above the river bed. A flood gate, an intake and a spillway have also been provided.



Sectional Elevation of Power House.



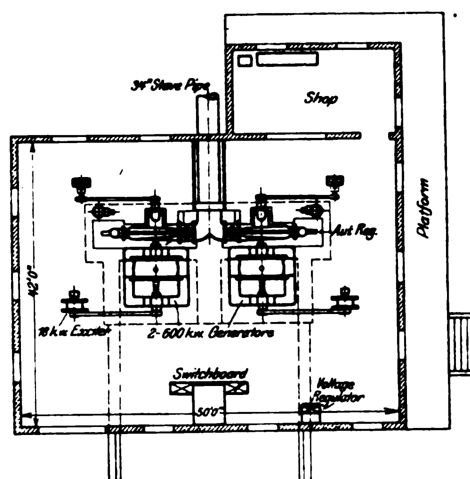
Section through Power House.

Fig. 375.

Hydroelectric Plant

of

Ketchikan-Alaska.



Plan of Power House.

Pipe line. This consists of 1500 ft. of 40 in. and 1050 ft. of 34 in. wooden stave pipe. The bands are spaced one and a half inches apart at the lower end of the pipe giving a factor of safety of 4.

Power house. The building which is 50 ft. × 42 ft. is a frame structure, resting on reinforced concrete columns. There are two 800 single horizontal turbines in cast iron spiral casing and with quarter turn discharge. They are directly connected to 600 kw., 3-phase 6600 volt 60 cycle generators. The exciters are of 18 kw. capacity and generate current at 120 volts. These are belt driven machines. See fig. 375.

Transmission line. Consists of one three-phase circuit, the conductors being of aluminum wire having the equivalent conductivity of No. 2 B. & S. copper wire. The circuit is stepped down to 440 volts by means of three 250 kw. water-cooled transformers.

206. Great Western Power Development. California.

(Head 419 ft.)

This plant is situated on the North Fork of the Feather River, near Big Bend, California. The ultimate capacity of the plant will be 144,000 HP. with an operating head of 520 ft.; at present the plant's capacity is 55,000 HP under a head of 419 ft.

The minimum flow of the river is 1200 sec. ft.

Dam. In carrying out the first part of the development, a temporary timber diverting dam with a maximum height of 25 ft., and 316 ft. long, was built across the river 600 ft. below the intake tower. This is being

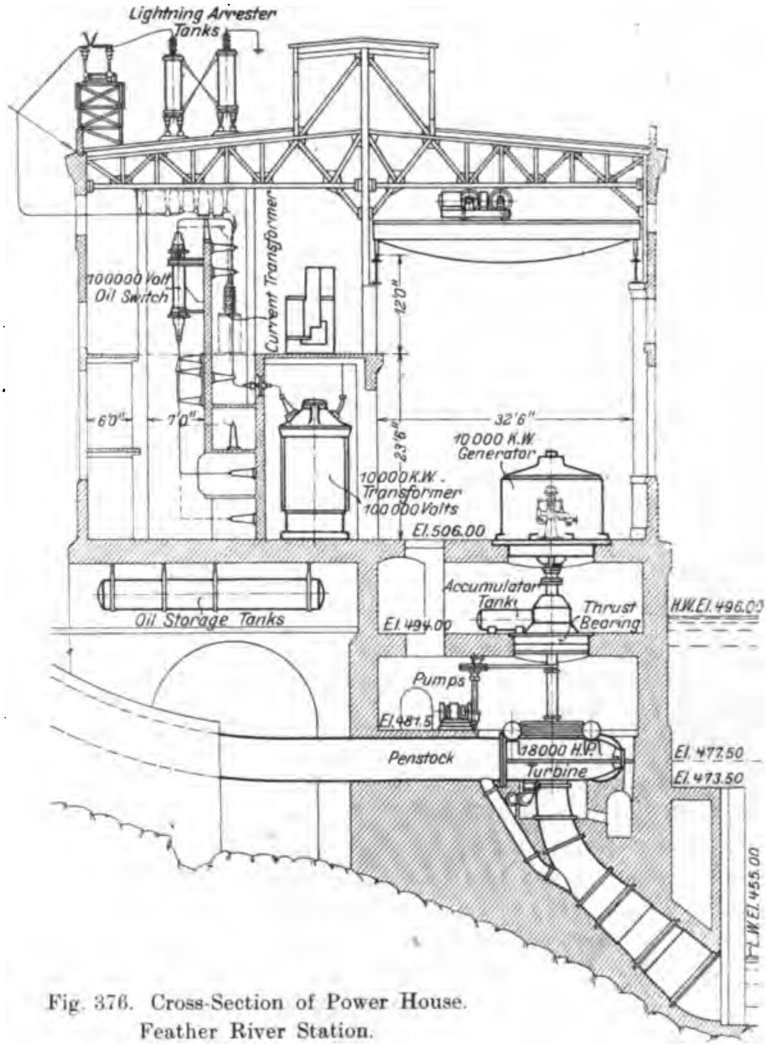


Fig. 376. Cross-Section of Power House. Feather River Station.

replaced by a concrete dam rising to about 129 ft. above the water level. It will be 500 ft. long at the top, thereby creating a storage reservoir of 400,000,000 cu. ft. capacity. The concrete intake tower, provided with gates and screens has been erected at the tunnel entrance. This tower consists of a circular concrete well 28 ft. inside diameter with twelve radial walls projecting from the outside surface accomodating 48 gate openings, these being arranged in four horizontal rings.

Tunnel. Water is conveyed to the power house by means of a tunnel 15,168 ft. long; 11,768 ft. of which is an old tunnel which has been enlarged to the proper cross section area. The grade of the old tunnel was $\frac{1}{2}$ per cent and the new one has been driven on a drop of 1 in 3000. From the tunnel the water enters a steel header, the cross-section of the tunnel being gradually changed from the normal to a circular shape. Emerging from the tunnel, the 16 ft.-9 inch header turns to an angle of $46\frac{1}{2}$ degrees and continues on a tangent for 112 ft. connecting at its end with the surge pipe. Two drains, two man-holes and air valves have been provided.

Pressure pipe. There are five pressure pipe connections, four of which correspond to the main units and one to the exciter. The main unit pipes are 5 ft. in diameter and the exciter pipe is two feet in diameter. They are about 600 ft. long. A cast steel gland expansion joint is provided at the upper end of each pressure pipe.

Power house. The power house a cross-section of which is shown in fig. 373, is 185.5 ft. long and 74.5 ft. wide, and accomodates 4 turbines of the inward flow Francis type, operating at a speed of 400 r. p. m. designed to develop 18,000 horse power at 520 ft. head. The direct connected generators have the following charateristics: 10,000 kw., 11,000 volts, 60 cycles and 3-phase. There are two exciter sets, consisting of 500 HP impulse wheels, each direct connected to a 250 kw. 250 volt direct current generator. The current is stepped up to 100,000 volts by four 10,000 kw. 3-phase water-cooled, oil-insulated transformers. These are delta connected.

207. Grace Plant of the Telluride Power Co. Idaho.
(Head 440 ft.)

This large and important plant is located at Grace, Idaho, and derives its power from the Bear River, 380 cu. ft. per sec. being required.

Dam. The diversion of the water is effected by a timber crib dam, 38 ft. high, 75 ft. wide and 340 ft. long with a 160 ft. spillway. The intake is cut into the dam, the bottom of it being 15 ft. below the crest of the dam.

Pipe line. The water then flows through a pipe line 23,221 ft. long, which terminates in a 50 ft. $\times$ 76 ft. standpipe above the power station. The upper part of the pipe line, 4157 ft. in length, is of wood stave construction, the diameter being 8.5 ft. The rest is composed of riveted steel pipe, the thickness of the plate being $\frac{1}{4}$ inch, and the diameter varying from 7.5 ft. to 8.5 ft. Air valves 10 inches in size have been installed at all high points. Four drain valves, from 8 to 10 inches in diameter have also been provided. The pressure pipe is of steel, the diameter varying from 7.5 feet at the upper end to 6.5 ft. at the lower, the plate thickness varying from $\frac{3}{8}$ inch to $1\frac{1}{16}$ inch.

Power house. The Grace station building is of concrete construction, 60 ft. wide and 140 ft. long. It accomodatés two turbo-generator units, each consisting of a horizontal shaft inward-flow reaction turbine, rated to deliver 8500 HP at 300 r. p. m.; these are directly connected to 5500 kw. 2300 volt 60 cycle 3-phase revolving field generators. Two 100 kw. exciters have been installed, each being driven by a 37 inch Pelton wheel. The permissible output of each exciter is more than 70% greater than

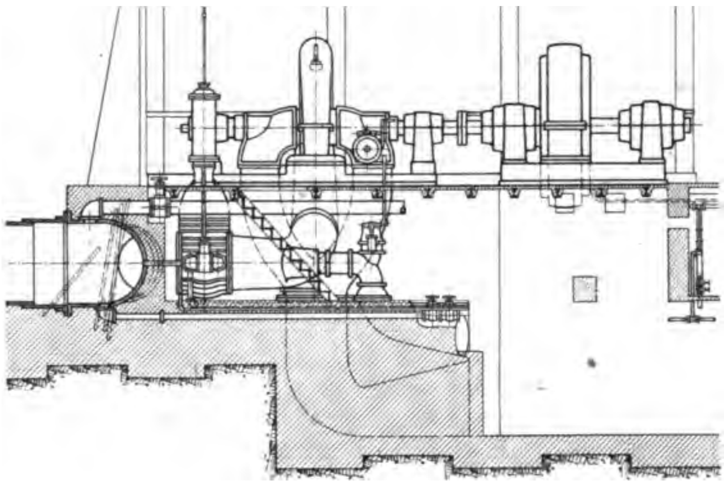


Fig. 377. Section Showing Generating Unit at Grace Station.

is needful for the two generators. Six 1833 kw. transformers with a seventh reserve unit, are installed on the basement floor, their ratio of transformation being 2300/51,000 volts. These transformers are of the water-cooled type, and are delta-connected on the low tension side and star-connected on the high tension side, and are arranged to allow operation at 88,000 volts. The energy is transmitted to the sub-station at Logan.

Fig. 377 shows a section illustrating a generating unit.

208. White River Development. Washington.

(Head 480 ft.)

This important development is situated about 20 miles East of Tacoma, Washington.

Dam. From the diversion dam, the water is carried through an earthen ditch about five miles long to the main storage reservoir. This holds enough water to operate the plant for a period of 3 or 4 months should no additional water be received. It is made up of several existing lakes, which are inter-connected and which have their capacities largely increased by heavy embankments and masonry dams. The flow line of the reservoir is about 4 miles long.

Tunnel and pipe lines. From the reservoir the water is carried through a tunnel about 3000 ft. long to the head basin. At present only one tunnel has been built, but eventually another will be added. From the

head basin a separate steel pipe line has been built for each turbine. These lines are approximately 2200 ft. long, 8 ft. in diameter at the upper end and 6 ft. in diameter at the power house end.

Power house. This is designed for a total length of approximately 373 ft. About 45 ft. of its width is used for the generating units, which are placed along this section with their shafts in line. Room has been provided for 6 units, although only two are installed at the present time. The other side of the building, 26 ft. wide has been given over to the exciter units, pumps, toilet rooms and transformer bays.

The two main units have a maximum capacity of 20,040 HP each, under a head of 480 ft. They run at 360 r. p. m. Each turbine is direct-connected to a 60 cycle, 3-phase 6600 volt generator.

The turbine runner is of the high pressure Francis type with horizontal shaft. Water is admitted to the runner through a cast steel spiral casing.

The runner which is also of cast steel, divides the water into two lines of flow, and then discharged from the wheel by two quarter turns to separate draft tubes. The flow of water to the turbine is controlled by a cast steel butterfly valve.

The spiral casing of this turbine is, the makers believe, the largest steel casting of this kind ever made. The butterfly valve is over 7 ft. in diameter, and the makers believe it also to be the largest valve of this type ever used. The shaft carrying the turbine runner is nearly 2 ft. in diameter and the bearings for it are about 16 in. in diameter. All parts of the turbine which have to withstand the operating pressure of the water have been tested in the shops under a pressure corresponding to about 900 ft. head. To give an idea of the size and strength of the parts it is only necessary to state that the total testing load on the butterfly valve has been about 1,500,000 lbs.

The governors for these turbines are of the Allis Chalmers standard oil pressure type, and the specifications call for extremely close speed regulation. Separate governors are supplied for each unit but a rather elaborate central oil pressure system has been employed.

As this plant has to regulate the supply of energy to the whole system of the Pacific Coast Power Company, the problem of proper regulation is an important one. It becomes of still greater importance on account of the necessity of conserving the storage supply of water. To meet this condition, and handle the water in a way which will protect the plant from injury and at the same time save water, pressure regulators have been installed. These are similar in design to those the Allis Chalmers Company has installed on the 18,000 HP turbines in the Great Western Power Plant, a design which has done excellent service on this and other installations.

One exciter is provided for each three units of the ultimate equipments. As the proportionate amount of the water which these wheels use is not large, these are of the impulse type, and governed by means of a deflecting hood similar to that with the exciter units of the Kern River No. 1 Station of the Edison Electric Company, built by Allis Chalmers Co.

209. Kern River Plant. California.

(Head 870 ft.)

The usual minimum flow of the Kern River is 400 cu. ft. per sec., and although in the dry season a flow of 200 cu. ft. per sec. has been recorded, the installation was built on a basis of 470 sec. ft., as a steam auxiliary has been provided and the average flow of the river is something like 800 sec. ft.

Dam. This structure was built of Cyclopean concrete, and its height was calculated to give a depth of water of about 30 ft. back of the dam. The length of the dam on top is 203.5 ft. while the bottom length is only 52.8 ft.

A drainage tunnel 365 ft. long is used to clean the reservoir of silt. The entrance to it is controlled by two steel gates 9 ft. $\times$ 3 ft. 1 in. They are operated by hydraulic cylinders under an oil pressure of 200 lbs. per sq. in.

Water is conveyed to a regulating reservoir by means of gravity tunnels inter-connected by open flumes. The total length of the conduit is 44,945 ft. of which 42,910 ft. consist of 19 tunnels. The velocity of the water is 9 ft. per sec. The regulating reservoir measures 30 ft. $\times$ 42 ft. with a depth of 8 ft. It is lined throughout with heavy reinforced concrete and contains the regulating gates and screens through which the water passes to the header. A spillway is also provided, and this is controlled by flash boards.

Pressure pipes. The header is 7.5 ft. in diameter; the pressure pipes leading to each wheel is 28 inches in diameter whereas the exciter penstocks are 10 inches in diameter. On each main pressure pipe two valves are placed, one hand controlled and the other electrically controlled.

Power house. The building is 164 ft. long and 66′-6″ wide and contains four units with a total capacity of 40,000 HP. The speed of the Pelton wheels is 250 r. p. m. and as a single wheel to develop the power of each unit would have been objectionable as to size, each hydraulic motor is composed of two wheels the generator being inter-coupled.

The regulation is obtained by the deflection of the jet, this being controlled by the needle sliding in the nozzle. Esher-Wyss oil type governors operating under a pressure of 125 lbs. were adopted for the movement of the nozzle, the time of deflection to be one second. It was desired also to obtain a speed regulation of 5% when full nominal load was instantaneously thrown off with a corresponding regulation with the addition or deduction of partial loads. Such regulations would be equivalent to a change in speed of 6.36% when 25% overload was thrown off, or 7.64% when 50% overload was lost. The rotor was therefore designed to take care of these effects, and the following formula was used:

$$s = 8 \times 10^5 \frac{P \times t}{r^2 \sum W R^2}$$

in which

$s =$ % change in speed,
$P =$ brake HP thrown off,
$t =$ time in seconds for nozzle deflection,
$=$ 1 in this case,
$r =$ revolutions per minute,
$W =$ weight in lbs. of revolving parts of unit, and
$R =$ radius of gyration of revolving parts

in the present case, $\sum R^2$ is 1,800,000 lb. ft², and this was obtained by increasing the weight of the generator field spider.

The generators are of the revolving field type, three phase 50-cycle current being generated at 2300 volts. Their normal rating is 5000 kw.

The exciters, of which there are two, are 250 kw., 125 direct current machines, with a speed of 450 r. p. m. They are direct connected to impulse wheels having automatic oil pressure governors and stationary needles in the nozzles. The station also contains four banks of three 1667 kw. trans-

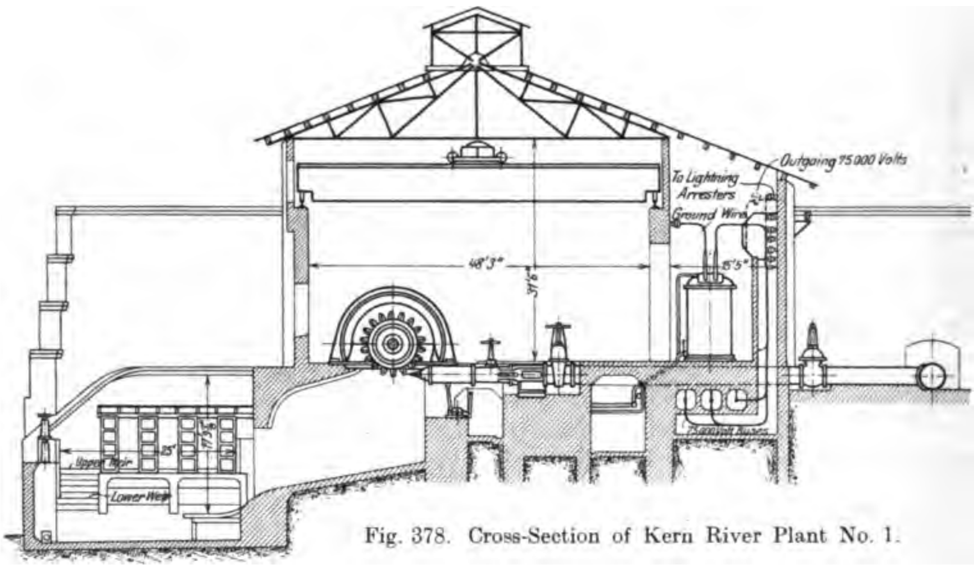


Fig. 378. Cross-Section of Kern River Plant No. 1.

formers each. These are delta-connected to the generators and star-connected to the transmission lines. The voltage is raised to 75,000 volts.

The switchboard is mounted on a gallery near the center of the power house where the attendant has a clear view of the whole generator room.

A cross-section of the Kern River plant No. 1 is illustrated in fig. 378.

Transmission line. The length of it is 117 miles and it consists of three separate circuits. The conductors are of hard drawn copper, and are composed of seven strands. Steel towers are used and spaced every 700 ft. The insulators are of the pin-type, and $18\frac{1}{4}$ inches high.

210. Plant of the Rio Janeiro Light and Power Co.

(Mean Head 1000 ft.)

Dam and intakes. The dam is a concrete structure, having a length of 656 ft. along the crest, and a maximum height of 138 ft. The effective available head is derived from three conditions occurring as follows: a natural fall of 285 ft. exists in a distance of 1000 ft; then a series of rapids in a distance of 6000 ft. give an additional head of 600 ft; and finally the dam increases the head by an amount varying from 69 to 131 ft., according to the surface level in the reservoir.

The intake is arranged in such a manner as to permit taking the water at three levels as the water elevation in the reservoir will allow. Screens and gates are provided at the three levels, and the location of the lowest intake entrance is such that the head will vary by 63 ft. between reservoir full and reservoir empty.

Pipes lines. The intake leads into two riveted steel pipes, 8 ft. in dia., supported by concrete piers 24 ft. apart. The actual maximum pressure head in the pipe is 57 ft. and the thickness of the pipe shell varies from $\frac{1}{4}$ in. to $\frac{5}{8}$ in. Each feeder is 1683.8 ft. long and its entrance is controlled by a 9 ft. $\times$ 12 ft. Coffin gate. The disposition of a double pipe line thus permits the use of one half of the hydro-electric system in connection with one feeder, while the second feeder operates the other half independently.

The pipe line leads through a series of tunnels and at the end is provided with a relief pipe and valve house. From this valve house, 6 pressure pipes, 3 ft. in dia. lead to the main units in the power house, and one pipe 12 in. in dia. leads to the exciter set. These seven headers are of lap-welded steel with flanged ends held together by bolted clamping rings.

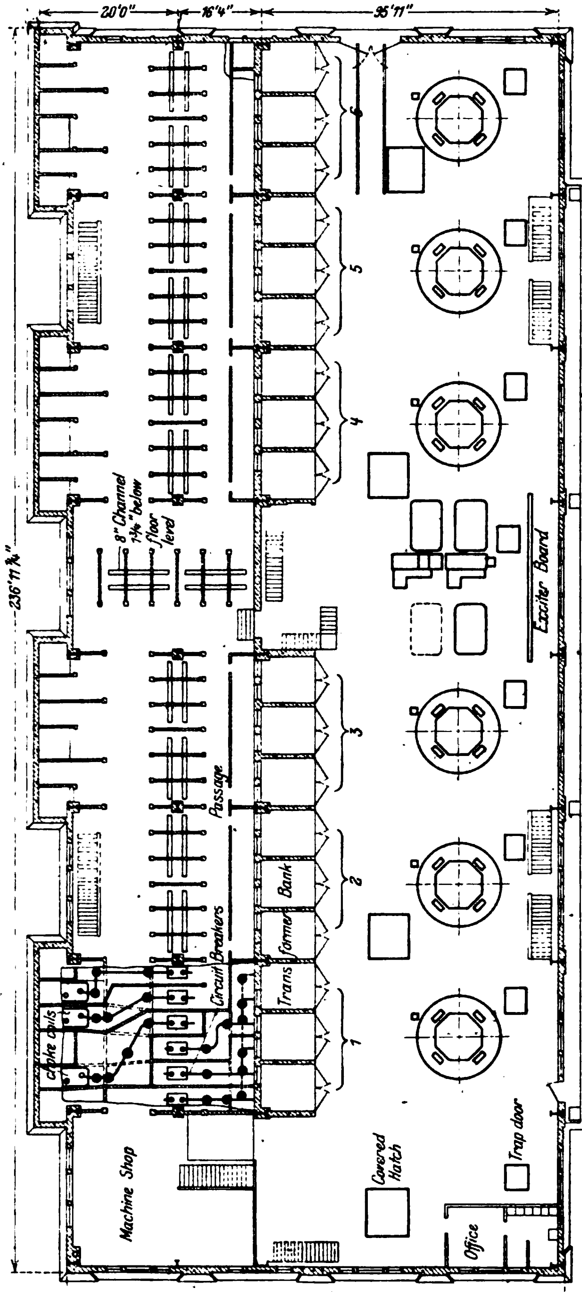
Power house. The power house, fig. 379, consists of a steel frame building with brick walls finished with cement plaster. The building is 235′-7″ long, 95′-11″ wide and 78′-11″ from wheel pit floor to eaves. The basement walls and foundations are of concrete. The normal output of the plant is 40,000 HP and consists of six main units developing 8700 HP each giving a maximum unit capacity of 52,200 HP of which one unit is considered a reserve.

Turbines. These are of the Pelton type, and each has one single runner upon which streams are directed from 4 nozzles set at 90 degrees. The runners consist of cast steel disks to which 18 cast steel buckets are riveted. The fact that the maximum head is 1017 ft. and the minimum head (friction loss considered) is 898 ft., the ratio of peripherical speed to spouting velocity varies from 42.7 % to 45.4 %.

The governor is of the Escher Wyss type for high head work. To each turbine corresponds a governor pump, driven by a Pelton wheel supplied from a by-pass on the main turbine valve.

The guaranteed regulation in speed is 3 % speed change on 25 % load change, 6 % speed change on 50 % load change, and 10 % on 100 % load. The units run at 300 r.p. m.

Generators and electric equipment. The generator is of the internal revolving field type. The stationary armature is wound for a normal voltage of 6000 volts, and the field for an exciting voltage of 250. The normal rating of the generator is 4000 kw. and overload rating of 5000 kw. The exciters consist of three 200 kw. sets, wound for 250 volts and running at 500 r. p. m. There are six three phase banks of oil-insulated water-cooled transformers. The low tension winding is designed for 6000 volts and the high tension windings may give either 44,000 volts or 88,000 volts according to the kind of connections: multiple or series. The full load efficiency of these transformers is 98.3 %. The electrical connections are shown on fig. 380.



General Plan of Power House at Generator Floor Level.

379a. Hydroelectric Power Plant of the Rio de Janeiro Tramway, Light and Power Company.

211. Plant of the Nevada Mining and Milling Company.

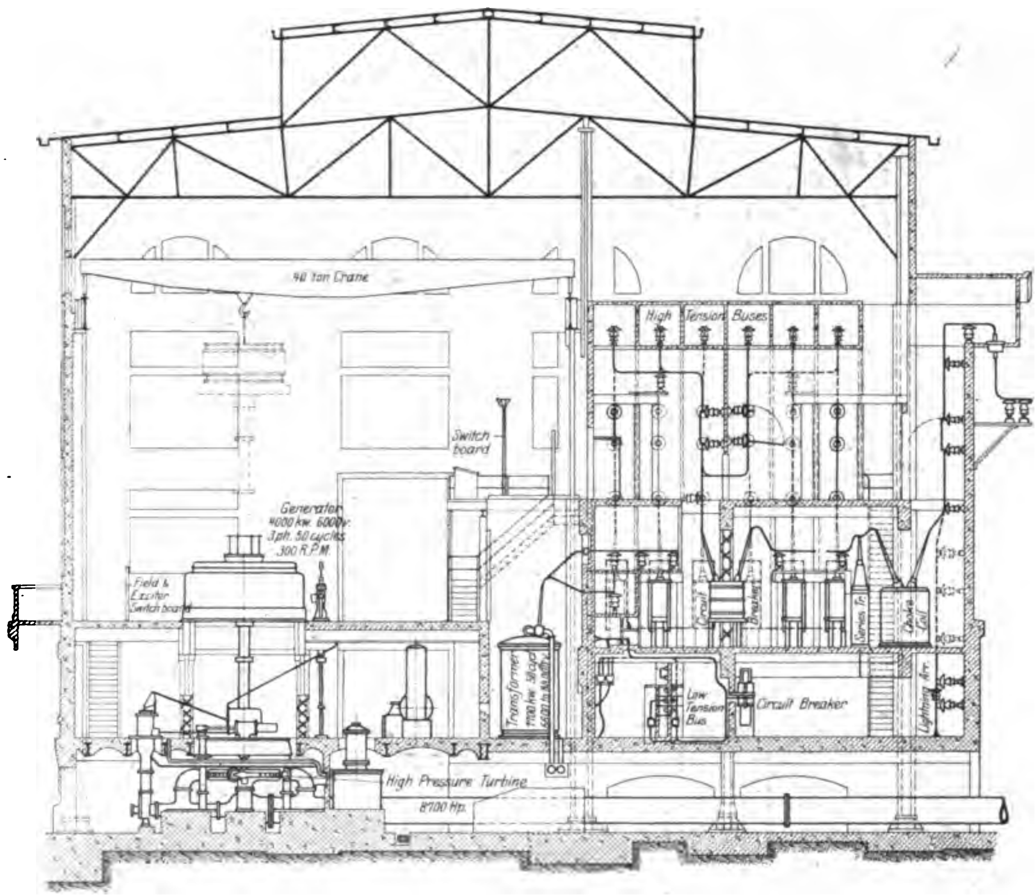
(Head 1068 ft.)

This plant is located on Bishop Creek, near Bishop, California.

Dam. A small diverting dam has been constructed which conveys the water to a pipe line, 12,000 ft. in length. The pipe line is composed of 3

sections: 6700 ft. of 42 in. wood stave pipe; 2150 ft. of 30 in. wood stave pipe; and 3150 ft. of 24 in. steel pipe. The latter resists a static head of 1068 ft. at the power house.

Power house. This is equipped with two 750 kw., 60 cycle, 2200 volt 3-phase alternators running at a speed of 450 r. p. m. and a 1500 kw. generator of the same characteristics, running at 400 r. p. m. The exciters are two in number, of 60 kw. output each and delivering 140 volt



Sectional Elevation of Power House.

Fig. 379b. Hydroelectric Power Plant of the Rio de Janeiro Tramway, Light and Power Company.

direct current. The water wheels are of the Pelton type. The two 750 kw. wheels have Sturgess governors; the 1500 kw. unit has a Lombard type Q governor.

Transmission line. It is 113 miles long and in crossing the White Mountains reaches an elevation of 10,500 ft. The conductors are of stranded aluminium, equivalent to No. 0. copper, and energy is transmitted to Tonopah and Goldfield, Nev.

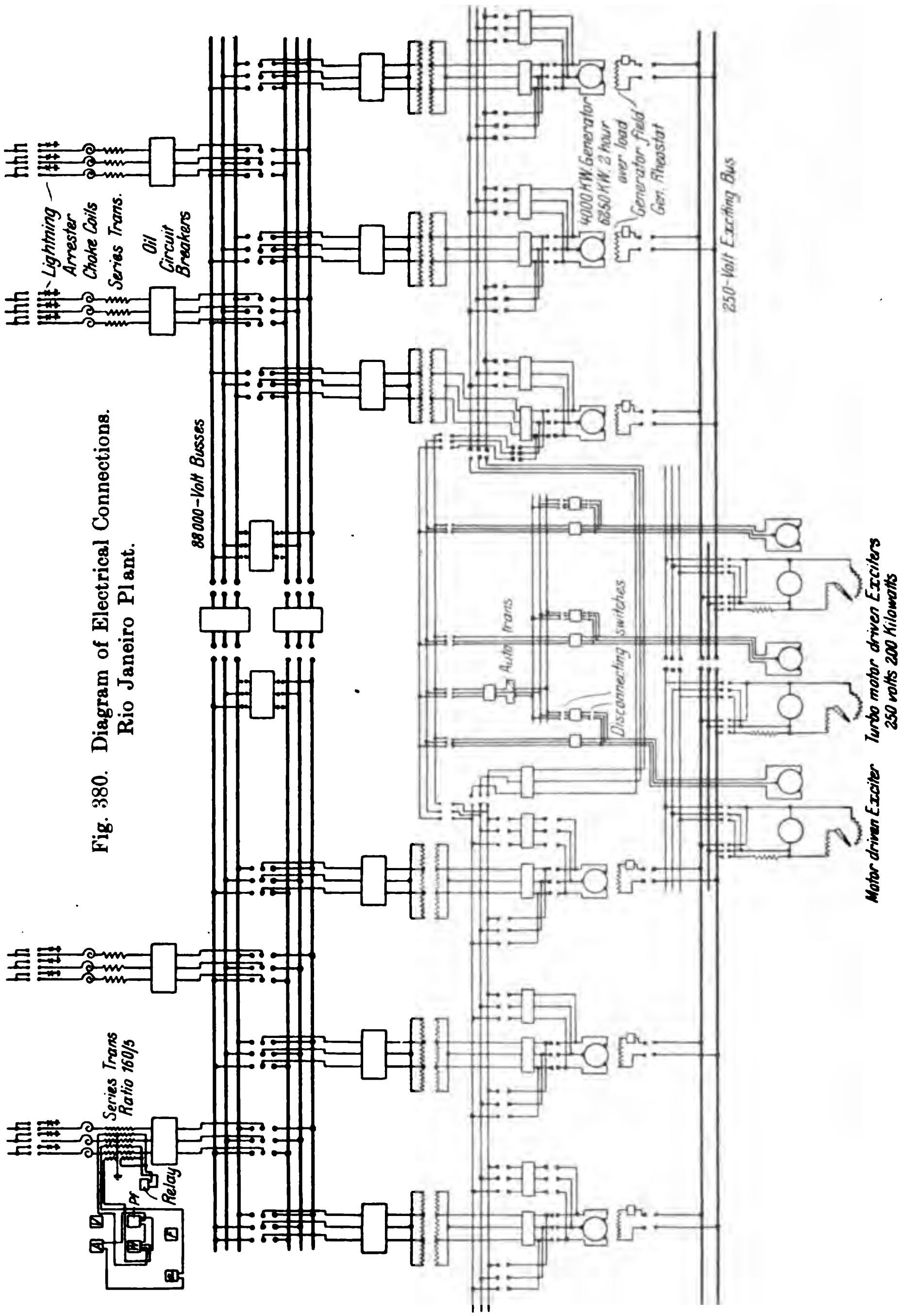


Fig. 380. Diagram of Electrical Connections.
Rio Janeiro Plant.

212. The Necaxa Plant of the Mexican Light and Power Co.

(Head 1400 ft.)

At present this is the most important development in Mexico. About 90 miles northeast of the city of Mecixo, the Necaxa, Tenango and neigh-

boring rivers have cut through the mountains surrounding the elevated plateau of Mexico, and fall abruptly to the low country bordering the Gulf. The rivers come together below, within a few miles, and eventually empty into the Gulf of Mexico. There is a series of picturesque falls with a drop of 4500 ft. in 10 miles. The map, fig. 381, shows the general scheme of development. The water of the Tenango is diverted into the valley of the Necaxa by a 40 ft. dam and a tunnel 12 ft. wide, 9 ft. high and 3000 ft. long. This tunnel lined with concrete wherever there is not solid rock, is constructed with a slope of 0.004. This gives a capacity of 875 cu. ft. per sec., which is sufficient to carry all but the extreme flood waters of

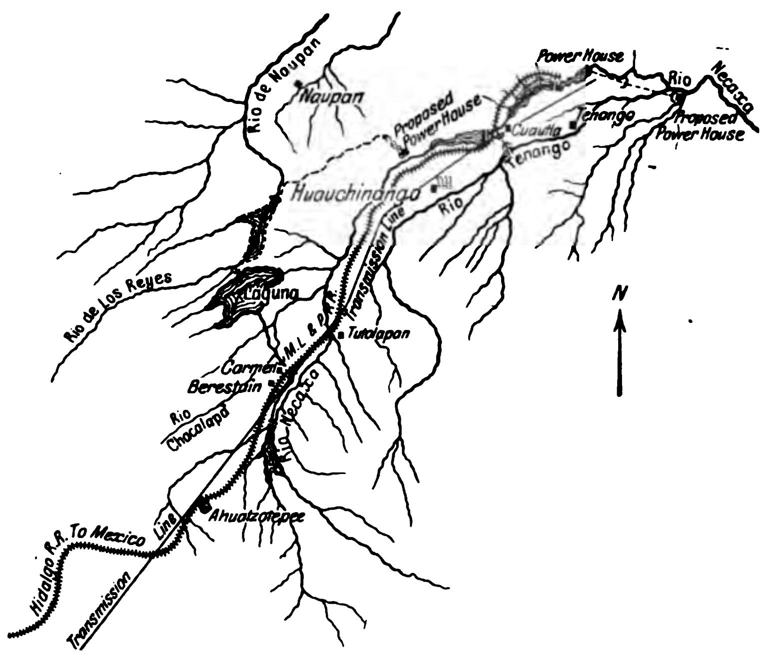


Fig. 381. Map showing Necaxa Development.

the Tenango. There are a number of storage reservoirs on the Necaxa, the most important being the Necaxa, Texcapa and Laguna Reservoirs. The storage capacity respectively of each is 1,585,000,000—642,000,000 and 700,000,000 cu. ft. The Laguna reservoir will eventually beincreased to a capacity of 2,450,000,000 cu. ft. by raising the dam after its drainage area has been increased by the diversion of the Rayos river into it. It is estimated that this storage amounting to 4,600,000,000 cu. ft. will be sufficient, not only to equalize the entire run-off of the streams during any year, but to enable the average flow of a succession of years to be depended upon.

Dam. The dams were built of earth by the hydraulic method, utilizing heads up to 400 ft. The Necaxa dam is 180 ft. high, 1276 ft. long at the crest, with a thickness of 950 ft. at the base, and 54 ft. at the top.

The slopes are 3 to 1 on the upstream and 2 to 1 on the down stream faces. About 2,000,000 cu. yds. of material were required in its construction.

Intakes. The water is admitted to the pipe lines through the gates shown on fig. 382. In order to avoid the operation of gates and racks under such high pressure, two vertical stand pipes, each with five flood gates have been constructed, either of which will permit all the water required for power purposes to pass. By this arrangement it is not necessary to operate the gates or racks under a head of more than 26 ft.

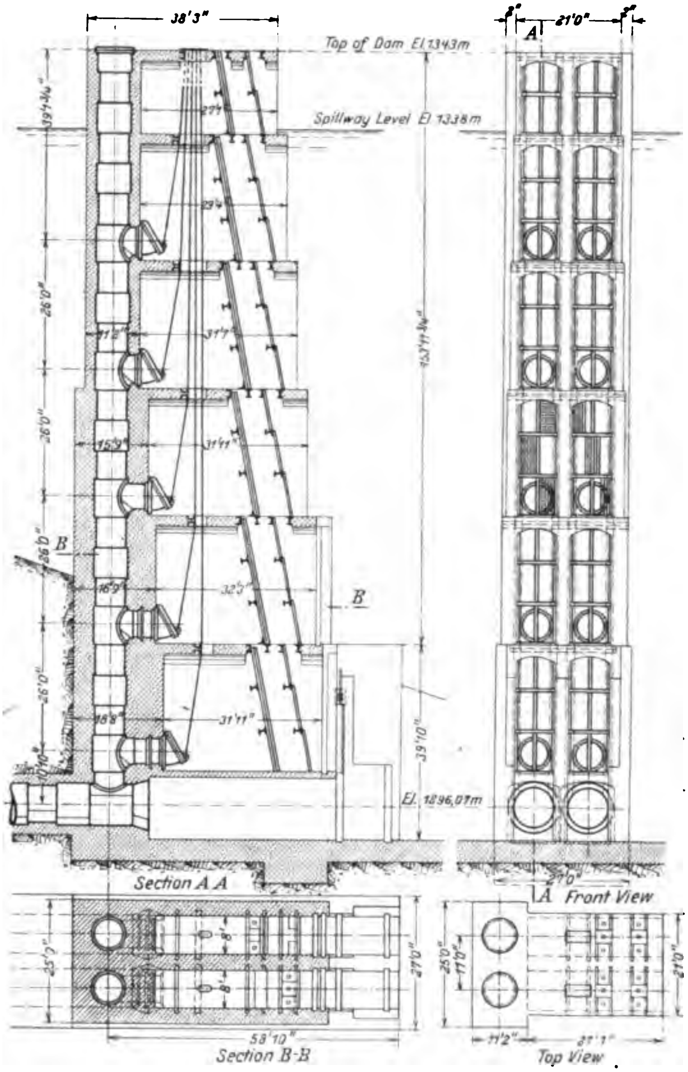


Fig. 382. General View of Intake for Dam No. 2. Necaxa Development.

Pipe lines. Two riveted steel pipes 8 ft. in diameter and $\frac{3}{8}$ inch thick pass through the Necaxa tunnel, with sluice gates discharging into the river. Below this point they are reduced to a diameter of 6 ft., and continue down stream a distance of 2200 ft., 800 ft. of which is through tunnels. Near the upper Necaxa Fall, these pipes are joined by 2 receivers each 22 ft. long and 7 ft. in diameter. From the receivers, 6 smaller pipes are carried down to the power house through two inclined tunnels. On each

of the 30 inch pipes, there are relief valves, 30 in. diameter, pipes from which lead up the hillside 310 ft. to an elevation above the crest of the dam. All pipes are connected with the receivers through valves and a central gate separates the two halves of the system, so that either half can be shut down without interfering with the other. The expansion joints are of the diaphragm type and are provided in each of the 6 ft. pipe lines. The velocity of the water is 7.5 ft. per sec. in the upper and 15 ft. in the lower pipe lines when the plant is running at full load. All pipes are sup-

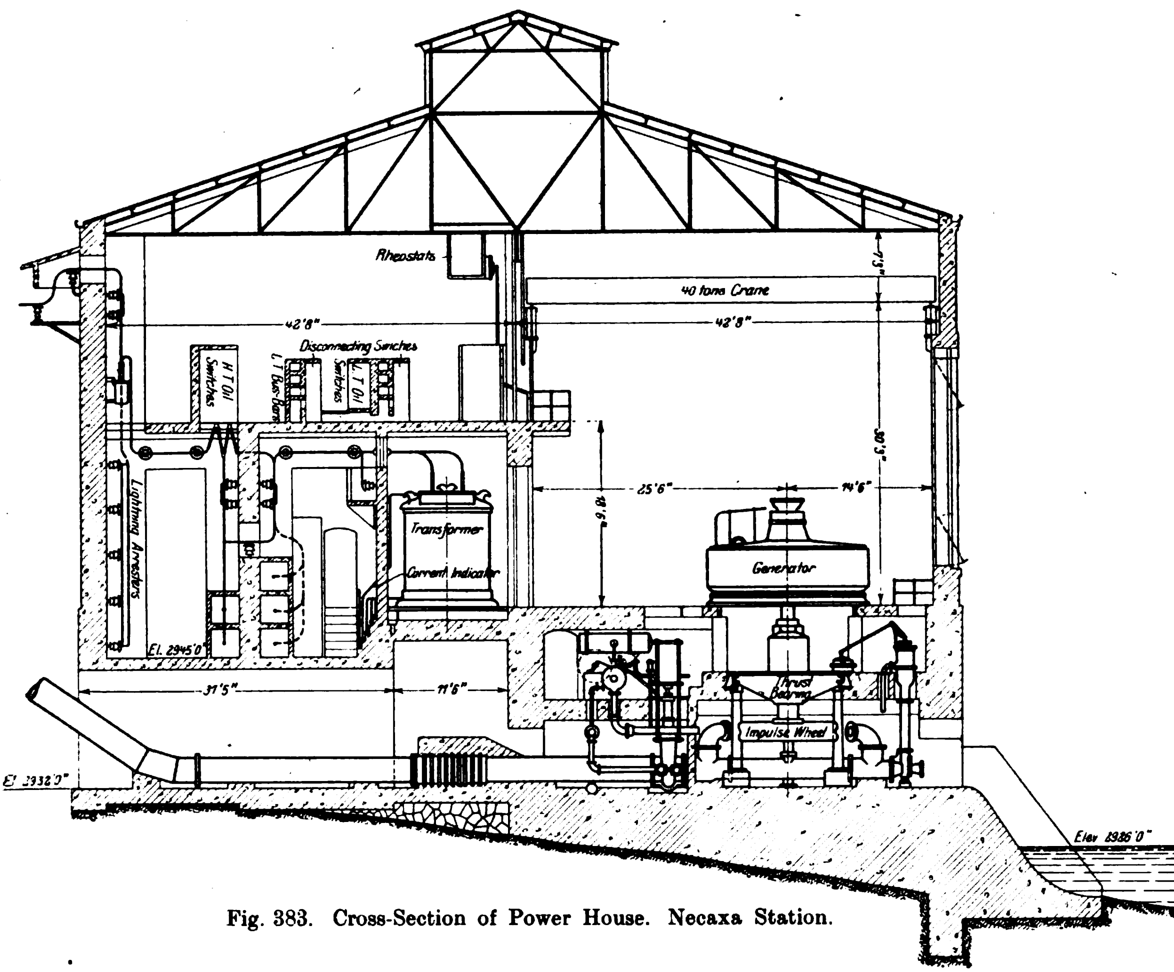


Fig. 383. Cross-Section of Power House. Necaxa Station.

ported on concrete piers throughout their length. The static head at the water wheel with the reservoir full is 1452 ft. which in service is reduced to between 1200 and 1300 ft. by the reservoir surface being lowered and by frictional and other losses.

Water wheels. The building, fig. 383, which is 235 ft. long and 88 ft. wide, contains six impulse wheels rated at 7000 HP, with a maximum capacity of 9000 HP making it possible to supply a peak load of 50,000 HP from the station. The wheels are of 100 in. pitch diameter, run at 300 r. p. m. and have solid cast steel center disks to which are clamped 24 steel buckets. Each has two $4\frac{1}{2}$ in. square, regulating nozzles, fixed on opposite sides

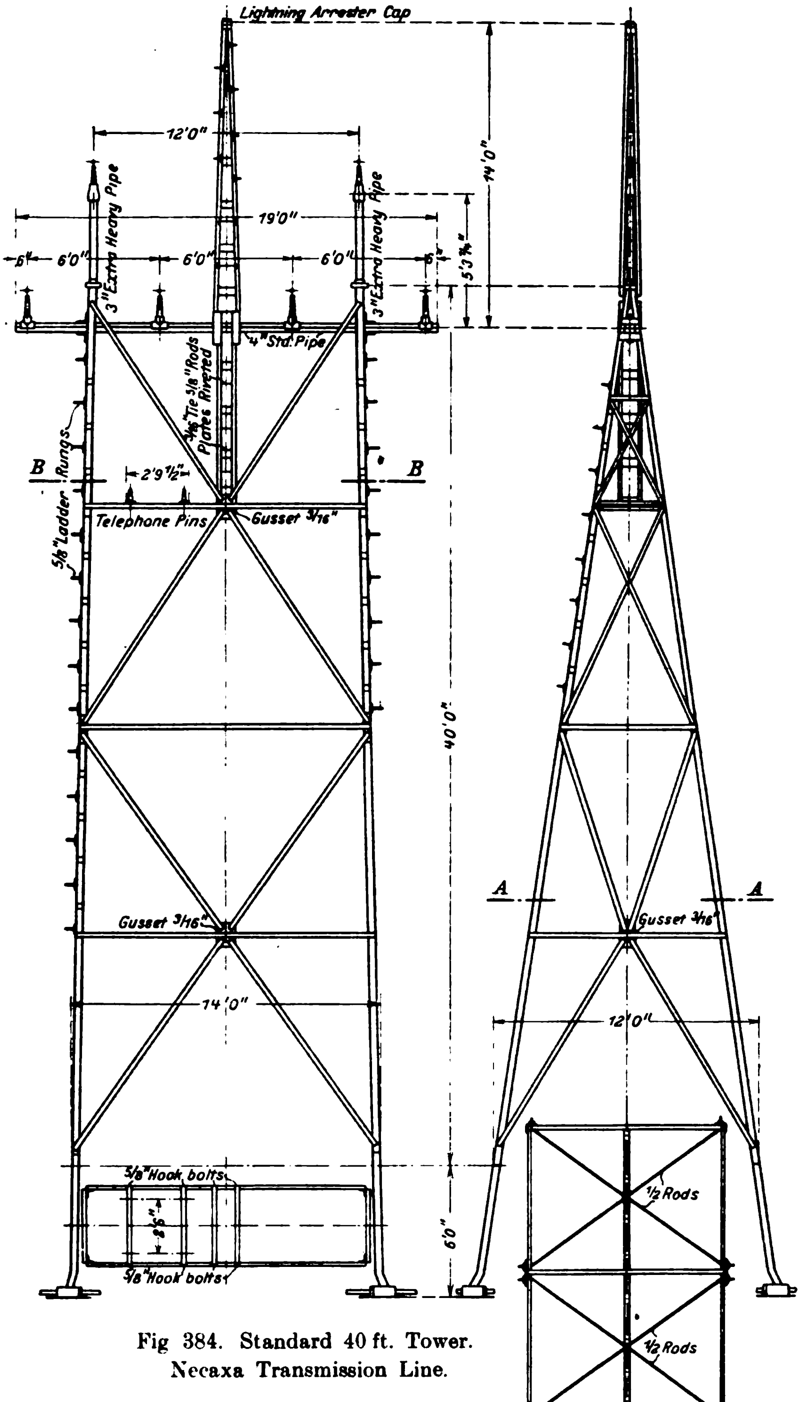


Fig 384. Standard 40 ft. Tower.
Necaxa Transmission Line.

of the wheels, but joined together so that they are opened and closed simultaneously. These machines were furnished by the Escher Wyss Co. of Zurich.

Generators. Each water wheel drives a 5000 kw. alternator of the revolving field type. These generate 3-phase current at 50 cycles 4000 volts. Two of the alternators are equipped with a 60 kw. direct-connected 125 volt exciter, but in ordinary operation, exciting current will be obtained from two 250 kw. 125 volt generators driven by induction motors wound for 4000 volts.

Transmission lines. There are two tower lines carrying four 60,000 volt circuits from the power house to the city of Mexico, and two circuits from that city to El Oro. The conductors are No. 000 B. & S. gauge wire, supported on porcelain insulators 14 in. in diameter. A standard 40 ft. tower is illustrated in fig. 384.

213. Hydraulic Power Development of Campocologno. Switzerland.
(Head 1450 ft.)

Campocologno is at about 3 miles from Lake Poschiavo, on the Poschiavino River, and not far from the town of Tirano, which is the terminal point of the Valtelina Electric Railroad. The Poschiavino river is a natural outlet of the above named lake, the drainage area of which is about 85 square miles. The concession stipulates that the average level of the water may be elevated 1 meter and lowered 7.4 meters; this difference of permissible levels, considering that the area of the lake itself is 500 acres, insures a storage of about 530 million cu. ft.

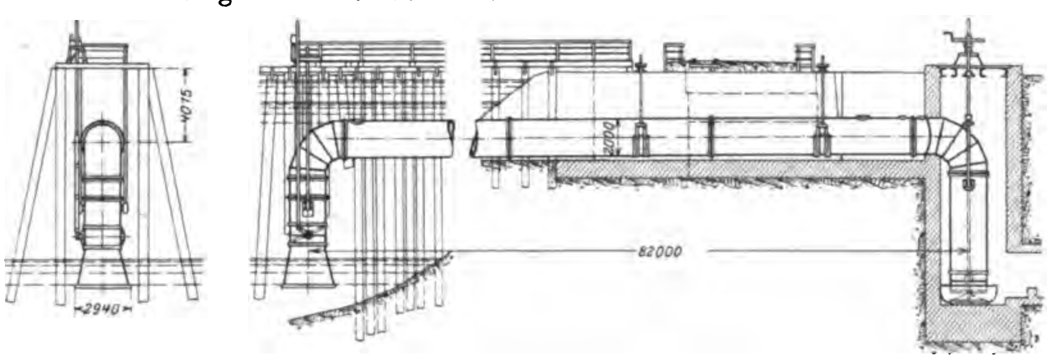


Fig. 385. Siphon forming Intake of Supply Canal of Campocologno Development.

Dam and intake. The dam is provided with 5 steel gates 13 ft. wide. The intake consists of a siphon, fig. 385, 270 ft. long, dipping in the lake and ending in a pit from whence a tunnel begins. The siphon, the diameter of which is 6.5 ft., extends about 200 ft. into the lake, being supported on trestles. The tunnel is of oval section about 8 ft. in height and width. In the forebay the level of the water is regulated by movable stop logs installed in one of the lateral galleries of the tunnel, permitting more or less water to accumulate or discharge. This forebay, fig. 386, is built of masonry, and its down-stream opening is pierced with 6 openings grouped together where the headers begin. A small building contains the gates of the pressure pipes and also their vent pipes.

Pipe lines. The inside diameter of the pipes varies from 2.8 ft. to 2.46 ft. approximately, and the shell thickness is nearly $\frac{7}{8}$ inch at the lower end. The velocity of the water inside is 11.5 ft. per sec. These pipes, the

lengths of which amount to about 3280 ft., descend in parallel vertical planes, have ten more or less sharp elbows which are anchored, and are supported at intervals by masonry piers. The Martin steel of which they

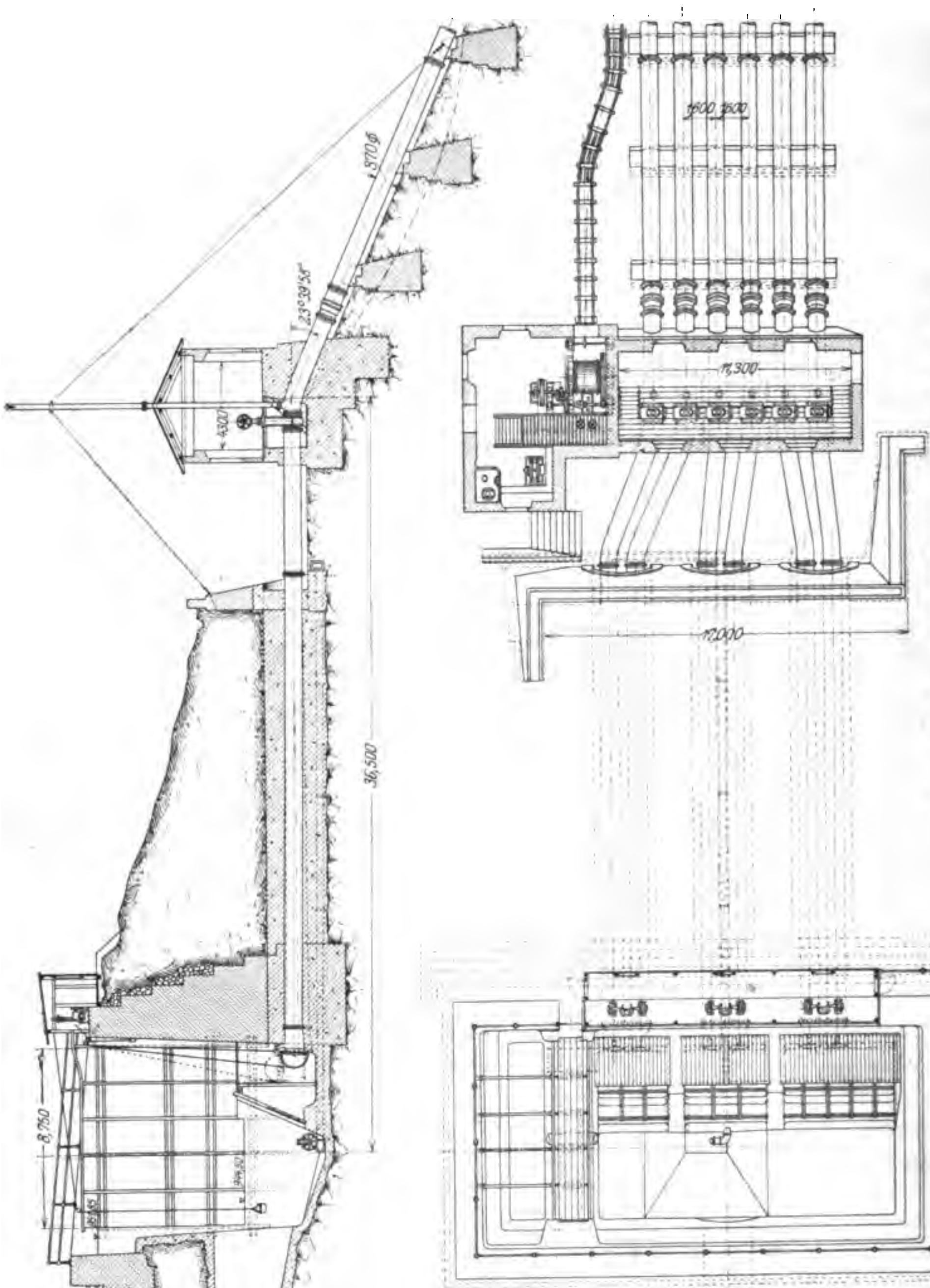


Fig. 386. Details of Forebay, Campocologno Plant.

are made, has an ultimate strength of 50,000 lbs. per sq. in., with an elongation of 25 %. A series of gates is provided close to the power house, with by-passes to prevent freezing.

Power house. This consists of a building 374 ft. long by 56 ft. wide the superstructure being of brick and its framework of steel. The capacity of the plant is about 40,000 HP and consists of 12 units of 3000 to 3500 HP each.

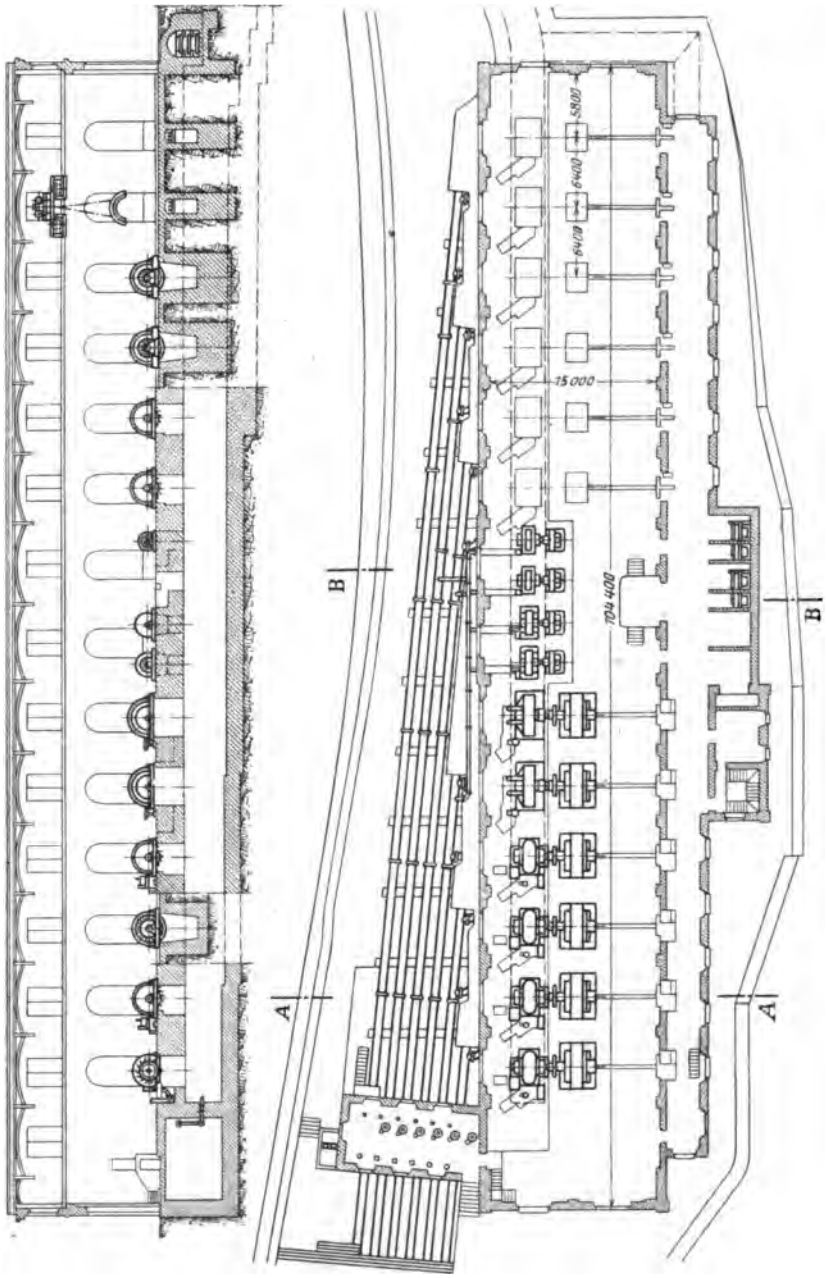


Fig. 387. Elevation and Plan of Campocologno Station.

Turbines. The turbines are of two types: Pelton wheels of Escher Wyss and Co. manufacture, and Girard partial admission turbines, made by Piccard and Pictet. The Pelton wheels have a sensitive regulator and an arrangement for discharge which dispenses with a heavy fly wheel;

this system is preferable when the variation of head is of little importance; the other turbines have slower regulators and heavier fly wheels, a system which is convenient for sudden and important variations of power.

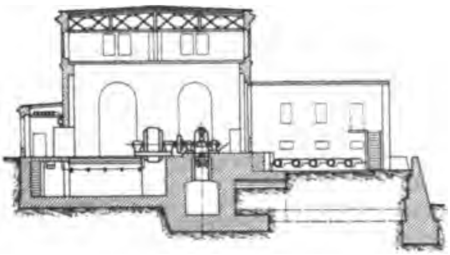


Fig. 388. Section A—A. (See Fig. 387.)

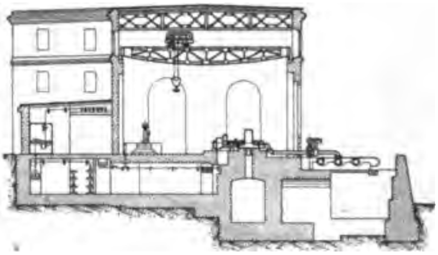


Fig. 389. Section B—B. (See Fig. 387.)

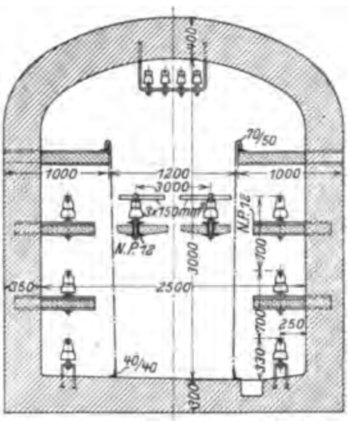


Fig. 390. Cross-Section of Cable Tunnel leading across Boundary, between Power House, Brusio, Switzerland, and Step-up Station, Piattamala, Italy.

Generators. The 3-phase alternators, the nominal capacity of which is 3000 kw. each at 375 r. p. m., generate current at 7000 volts, 50 cycles. The exciters, four in number, are shunt dynamo 150 kw. 115 volts at 430 r. p. m. Each of these machines is coupled to its turbine by means of an elastic coupling of the Zodel Voith type.

Transmission lines. There are two outgoing lines at 7000 volts which are carried in a small steel gallery across the river and then into a gallery of masonry, fig. 390, ending at a distance of 1640 ft., at the transformer station of Piattamala. Beginning at this point, the two 50,000 volt transmission lines are supported on steel towers. They follow the valley of the Adda up to Colico, then run along Lake Como to Bellano, and through Lecco arrive at the plants of Lomazzo, 90 miles, and Castellanza 105 miles. In the station of Lomazzo, the current is stepped down to 20,000 or 11,000 volts and distributed over several lines, one of which leads to the steam plant of Castellanza, which acts as a reserve to the plants of Tubigo, Vizzola and Campocologno.

214. The Boulder Hydroelectric Development. Colorado.

(Head 1830 ft.)

This plant was formerly designed as an emergency plant for the other plants owned by the Central Colorado Power Co., as it possesses a large water storage. The supply is obtained from the Middle Boulder Creek, the minimum flow of which is as low as 5 cu. ft. per sec. By means of the reservoir which has been created, and which has a capacity of 5,000,000 cu. ft., it is possible to regulate the flow to 37 cu. ft. per sec.

Dam and pipe line. The storage is secured by a masonry dam 172 ft. high. A 36 inch concrete pipe line leads from the reservoir 12 mile away.

to a second reservoir at the head of the pressure pipes leading to the power house. This concrete pipe is laid on a grade of 0.5 ft. per 100 ft., and is buried into 2 feet of ground. It was reinforced as per following table:

Number of Circumferential Bars in 3-feet Lengths of Pipe.

Head in ft.	Square bars	Head in ft.	Square bars
0 to 10	Four $\frac{1}{4}$ in.	50 to 60	Nine $\frac{1}{8}$ in.
10 „ 20	Six $\frac{1}{4}$ „	60 „ 70	Ten $\frac{1}{8}$ „
20 „ 30	Eight $\frac{1}{4}$ „	70 „ 80	Twelve $\frac{1}{8}$ „
30 „ 40	Eleven $\frac{1}{4}$ „	80 „ 90	Ten $\frac{3}{8}$ „
40 „ 50	Eight $\frac{1}{8}$ „	90 „ 100	Twelve $\frac{3}{8}$ „

In the sections where the static head ie above 100 ft., riveted steel pipe 39 inches in diamter and $\frac{1}{4}$'' thickness of plate, has been used, the plate being coated with asphalt.

Another pipe line carries the water from a second reservoir system to the power house. This line begins 37.5 ft. below high water line of the reservoir. At this point it is 56.5 inches in diameter and $\frac{1}{4}$ inch thick. The diameter decreases to 52 inches, then to 48 inches, and finally to 44 inches at the lower end.

Power house. The station building is a brick structure with steel frame, 113'-4'' long, and 45'-0'' wide. It accomodates two water wheels of the impulse type, each being rated at 10,500 HP. The governing is by deflection of the nozzles away from the wheel bucket, with needle adjustment in the nozzle for different constant loads. Each water wheel is direct connected to a 5000 kw., 4000 volt 60 cycles 3-phase generator. The transformers, located in one end of the power house raise the voltage from 4000 to 100,000 volts for the transmission of energy to Denver, 29 miles distant. At the other end are transformers which step the current up to 13,000 volts for the distribution of energy to the mining districts in the vicinity of Boulder. (Figs. 391—392.)

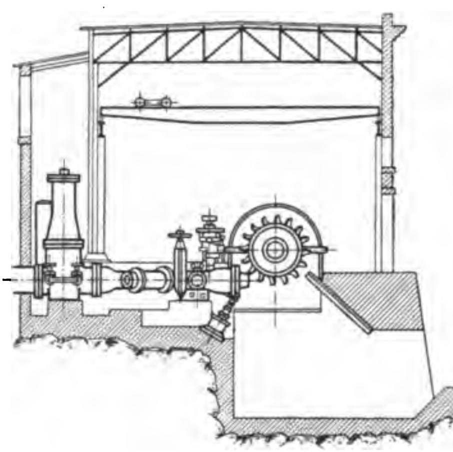


Fig. 391. Cross-Section of Power House. Boulder Plant.

215. Manitou Development. Pike's Peak. Colorado.

(Head 2350 ft.)

The hydroelectric station at Manitou is one of the most celebrated plants of its character in the world, the effective operating head being 2350 feet. The station is located in a gorge at the base of Pike's Peak, and derives its water supply from the public reservoirs of the city of Colorado Springs, the latter being located in the Pike's Peak range of elevations of from 9000 to 14,000 ft. above sea level.

Pipe line. Water is delivered to the plant through a steel pipe line about 17,000 ft. long, the pipe being installed in the following sections:

Upper line, spiral riveted, diameter 20 in., length about 4000 ft.; middle line lap riveted, diameter 21 in., length about 9200 ft.; and pressure line double butt strapped, triple riveted and belted flanges, metallic gaskets, diameter 20 in., length about 3800 ft. The hydrostatic pressure on the pipe line at the entrance to the station is about 1020 lb. per square inch, and the company has given unusual attention to the hydraulic problems involved in the installation, which was owned by the Pike's Peak Hydroelectric Company, prior to the recent purchase of that property by the Colorado Springs Light, Heat and Power Company.

Power house. The Manitou plant contains three 750 kw., 6600 volt, 3-phase, 60-cycle General Electric revolving field alternators, each being direct-connected to an 86 in. Pelton wheel having a normal speed of 450 r. p. m. Water is supplied to these units by a 45 deg. branch from the incoming pipe line, the supply pipe diameter being 10 in. and the nozzle diameter $2\frac{7}{16}$ in. There are also in service two 45 kw., 125 volt General

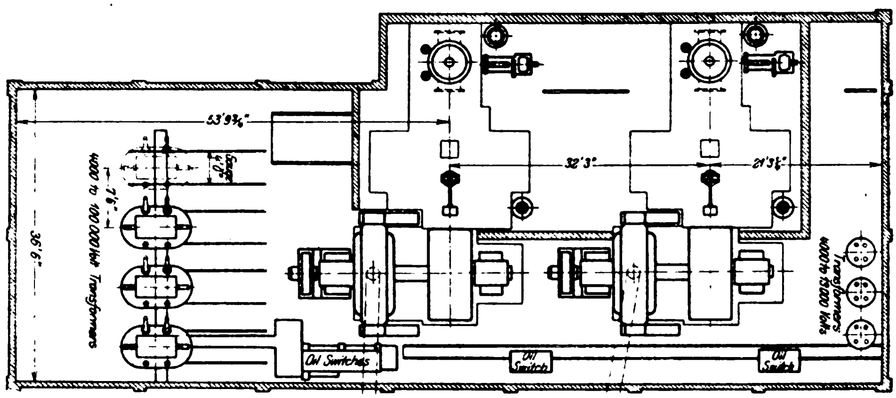


Fig. 392. Arrangement of Apparatus. Boulder Power House.

Electric flat compounded exciters, each being direct driven at 975 r. p. m. by a 42 in. Doble wheel, receiving water from the branch line equipment supplying the main units. The exciter wheel supply lines are 2 in. in diameter and the delivery nozzles are $\frac{3}{8}''$ in diameter. The pressure end of the pipe line is anchored to a concrete foundation between joints and the lower end is equipped with four relief valves set to open at from 1200 lb. to 1500 lb. per square inch, and discharging into the power house tail race through $2\frac{1}{2}$ inch pipes. Each main unit is provided with a gate valve, located in the supply branch, and driven by a $\frac{1}{2}$ HP 125 volt direct current motor. The company has found that single gate valves are the only type which are successful under the conditions of head utilized, it being practically impossible to keep either double gate or wedge type valves tight. The wheels are controlled by an oil pressure system, operated by Lombard governors and needle valves.

Transmission line. This is 3.5 miles long and is composed of two Nr. 000 circuits. The energy is transmitted at a pressure of 6600 volts.

The distribution system of the Colorado Springs Light, Heat and Power Co. is illustrated in fig. 393.

216. Hydraulic Plant at Vauvry. Switzerland.

(Head 3115 ft.)

This installation is situated on the left bank of the River Rhone a short distance above the point where that stream discharges into Lake Leman. The water is taken from Lake Tanay, at an elevation of 4644.5 ft. and is delivered to the wheels in the power plant at an elevation of 1528.8 ft. which represents a gross head of 3115.7 ft.

Pipe line. The head of the pipe line is at an elevation of 4559 ft. which is 65.6 ft below the normal level of the lake, and 84.2 ft. below its maximum level, and it terminates in a vertical shaft about 41.7 ft. deep. From the shaft a short gallery or tunnel 984 ft. long with a sectional area of 10.75 sq. ft. is built on an almost level grade. This tunnel is provided with bulk-

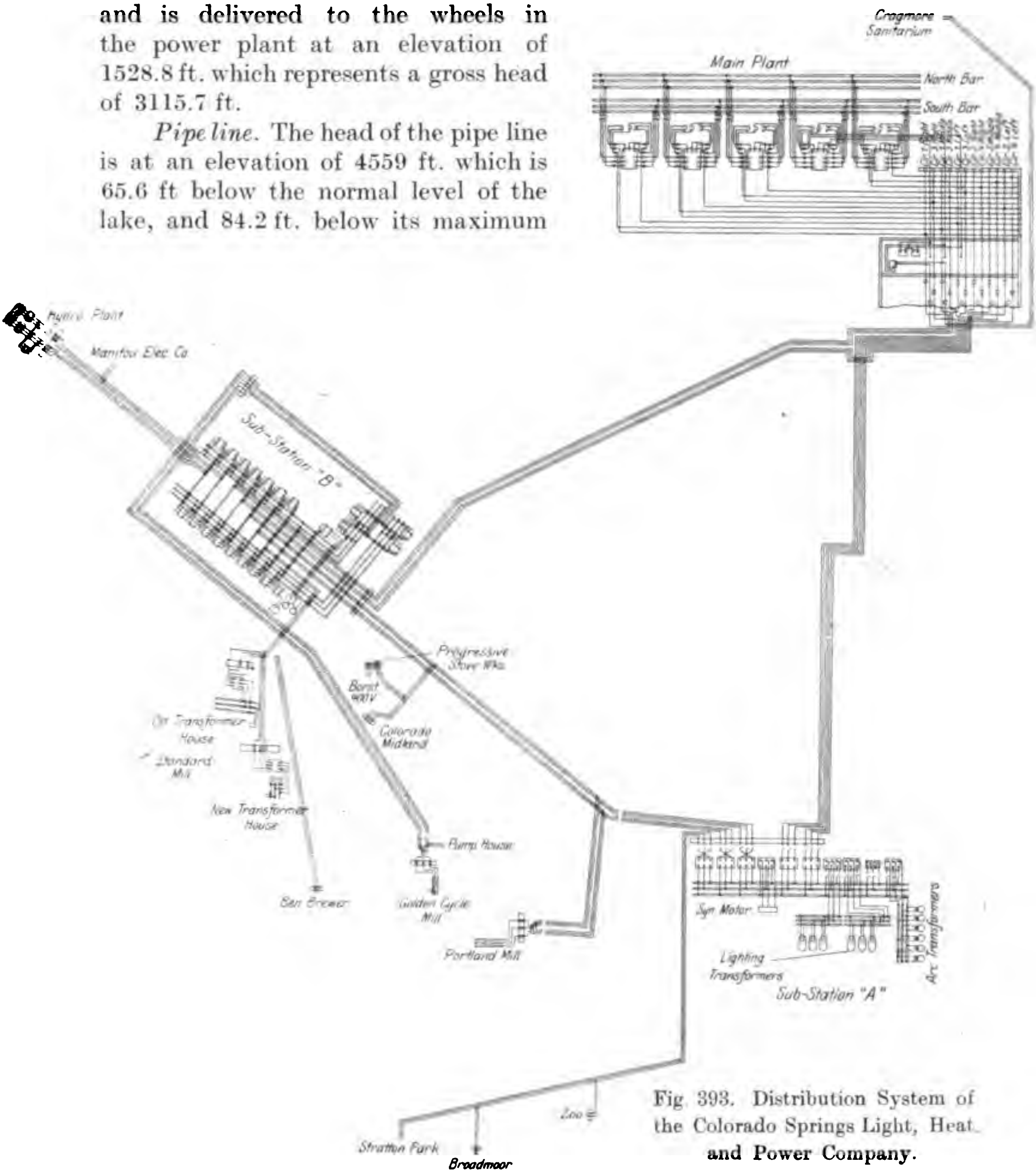


Fig. 393. Distribution System of the Colorado Springs Light, Heat and Power Company.

heads, pipes valves and other apparatus for regulating and controlling the supply of water taken from the lake; and at its end the pressure pipe line begins. For 328 ft., this line is a steel pipe 2.62 ft. in diameter; then for 984 ft., it consists of a masonry tunnel and finally for 3936 ft. it is again a steel pipe, 2.62 ft. in diameter. At the end of the last section, the pipe branches into 3 lines, each of 1.64 ft. diameter. One of these pipes extends to the power house plant but the others are plugged, and will be built to the power house only when the demand for power necessitates their construction. At the point of junction of the single and triple pipes the head of water is only 689 ft., but from here the descent is very abrupt. At this point also there is a retaining valve, and a vertical standpipe 1.31 ft. in diameter, and 82 ft. high which relieves the water hammer in the pipe line above.

The steep grade pipe-line, from the junction point just mentioned to the power house, is 6363 ft. long and has a fall of 2952 ft. The thickness of shell varies from about .275 to .45 inch, finally terminating in a Y, each branch of which is provided with a valve. From this Y, two 1.12 ft. pipes extend for a distance of 4264 ft., with shells varying in thickness from about .3 to .7 of an inch. The transverse joints are made as shown in fig. 394; and in order that the joints may be tight, a suitable gasket is inserted before tightening up the bolts. As the pipe line lies in a trench following the surface grades, it has many bends; and to provide for these, the wedge shaped pieces shown at *b* are inserted at the joints. The sections of pipe used varied in length from 16.4 feet to 32.8 ft., and weighed from about 1760 lbs. to about 2500 lbs. Each branch of the pipe line has at its lower end a slide valve provided with a by-pass, to permit it to be operated by hand.

Power house. The power house is a steel building 45.9 ft. by 216.5 ft. in plan. The two pipe lines described terminate underneath its main floor; and each supplies water to two wheels, all of which are of the impulse type. Two makes of wheels were installed, one of which is shown in Fig. 395.

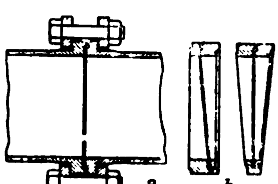


Fig. 394. Detail of Joint Connection for Pipe Line of Water-Power Plant at Vauvry, Switzerland.

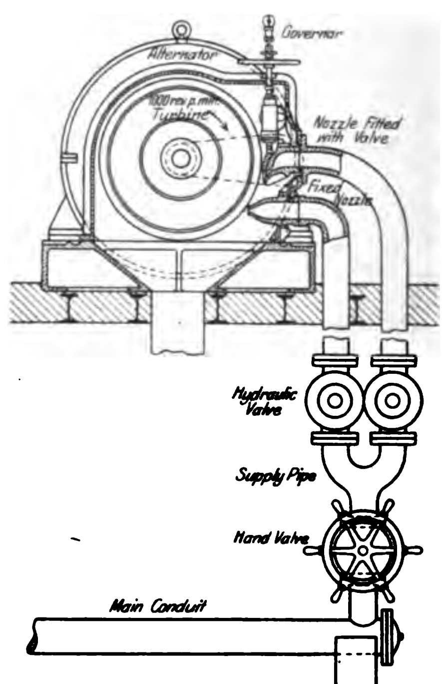


Fig. 395. Detail of Wheels supplied by the Société de Constructions Mécaniques de Vevey, for Vauvry Water-Power Plant, Switzerland.

Each wheel is supplied with water through two nozzles, one above the other, the upper one of which is provided with a device actuated by a governor, for controlling the supply of water. A vertical diaphragm divides the end of each nozzle into two openings, as shown. The pipe from which these nozzles are supplied rises vertically from the main, and is opened and closed by a hand valve, above which it divides into two branches, one leading to each nozzle and each opened and closed by a hydraulic valve controlled from the operator's platform on the floor above.

Each wheel is mounted on the shaft of an alternator, and operates at 1000 revolutions per minute. The dynamos are each of 500 HP capacity.

The installation is intended to supply electricity for lighting and power purposes to a large number of Swiss towns and villages in the valley of the Rhone.

217. Poglia River Plant. Italy.

(Head 4600 ft.)

A new 60,000 HP installation utilizing a water-fall of 4600 ft. of the Poglia River for the benefit of the city of Milan is now in construction. The Poglia is a tributary of the Oglia River, and rises in the Adamello mountains. The full head of 4600 feet is utilized in two steps. There are two stations, at Isola and Cedegola respectively. The station at Isola is fed with water through a conduit leading to a forebay, and a pipe system, which discharges into the turbines. The conduit has a length of one mile and is circular, 71 inches in diameter. Of the four pipe conduits projected two are now completed. The pipes begin with a diameter of 31.5 inches and a shell thickness of 0.28 inch; in the lower portion the diameter is diminished to 29.53 in. and 23.6 in. and the thickness of the shell increases to 1.26 in. The upper portion of the pipe is riveted, and the lower portion is lap-welded. The whole system is buried 8 ft. deep to prevent freezing. Seven generators, each rated at 7000 HP fitted with 500 HP exciters, are projected; four of these units are now being installed. The whole hydraulic part of the plant-pipes and turbines is being furnished by Escher-Wyss & Company and Professor Prasil of Zurich has designed a new regulation system for the turbines in order to meet the pressure fluctuations. The energy generated is transmitted at 12,000 volts to the lower station at Cedegolo. This station is fed both by the discharged water from Isola and by independent sources, and as there was no suitable space for a collecting reservoir on the precipitous rocks, the collecting basin was planned as a cellular structure in ferro-concrete, and consists of 140 cells arranged in three tiers one above another. From the basin two pipes, each 40 inches in diameter, lead down to Cedegolo, where 5 units, each rated at 4500 HP, have been put up. The main switchboards and transformers are at Cedegolo. The energy to be supplied to Milan will be transmitted at 72,000 volts by an overhead line, 75 miles in length. When this scheme is completed Milan will receive a total of 320,000 HP from hydroelectric plants.

Appendix.

The Water Powers of the United States. Legislation.

218. The Water Powers of the United States.

According to the latest census of the U. S. Geological Survey, the estimated aggregate stationary power, steam, water and gas, now in use in the United States, is probably over 30,000,000 horse-power[1]. The Geological Survey has estimated the national minimum flow water powers at 36,916,215 horse-power, and the aggregate, which it is estimated can be developed on a basis of 6 months flow, is nearly 67 000 000 horse-power.

To convey an idea as to the distribution of the enormous water power resources of the entire country, the reader is referred to Table LXXIII, taken from the Report of the Commissioner of Corporations on water power development in the United States, presented in 1912.

Table LXXIII. *Report of Commissioner of Corporations on water-power development in the United States, 1912.*

Potential water powers in the United States, by States, as computed by the United States Geological Survey and as revised by the Bureau of Corporations.

	Potential horsepower on basis of 90 per cent efficiency.		Potential horsepower on basis of 75 per cent efficiency.	
	Minimum	Assumed maximum	Minimum	Assumed maximum
United States	32,083,000	61,678,000	26,736,000[1]	51,398,000[1]
North Atlantic States:				
Maine	532,000	971,000	443,000	809,006
New Hampshire	162,000	295,000	135,000	246,000
Vermont	113,000	206,000	94,000	172,000
Massachusetts	142,000	273,000	118,000	228,000
Rhode Island	7,000	16,000	6,000	13,000
Connecticut	86,000	164,000	72,000	137,000
New York	1,244,000	2,037,000	1,037,000	1,698,000
New Jersey	53,000	127,000	44,000	106,000
Pennsylvania	331,000	821,000	276,000	684,000
South Atlantic States:				
Delaware	6,000	13,000	5,000	11,000
Maryland	51,000	146,000	43,000	122,000
District of Columbia	6,000	13,000	5,000	11,000
Virginia	590,000	1,144,000	492,000	870,000
West Virginia	457,000	1,261,000	381,000	1,151,000
North Carolina	693,000	1,050,000	578,000	875,000
South Carolina	552,000	812,000	460,000	677,000
Georgia	449,000	752,000	374,000	627,000
Florida	9,000	16,000	8,000	13,000

[1] Report of the U. S. Commissioner of Corporations. 1912.

	Potential horsepower on basis of 90 per cent efficiency.		Potential horsepower on basis of 75 per cent efficiency.	
	Minimum	Assumed maximum	Minimum	Assumed maximum
North Central States:				
Ohio	71,000	213,000	59,000	178,000
Indiana	52,000	141,000	43,000	118,000
Illinois	230,000	414,000	192,000	345,000
Michigan	216,000	352,000	180,000	293,000
Wisconsin	430,000	804,000	358,000	670,000
Minnesota	278,000	593,000	232,000	494,000
Iowa	192,000	458,000	160,000	382,000
Missouri	86,000	195,000	72,000	163,000
North Dakota	105,000	248,000	88,000	207,000
South Dakota	51,000	90,000	43,000	75,000
Nebraska	235,000	439,000	196,000	366,000
Kansas	130,000	323,000	111,000	269,000
South Central States:				
Kentucky	99,000	236,000	83,000	197,000
Tennessee	555,000	913,000	463,000	761,000
Alabama	611,000	1,132,000	509,000	943,000
Mississippi	38,000	75,000	32,000	63,000
Louisiana	1,000	2,000	1,000	2,000
Arkansas	26,000	73,000	22,000	61,000
Oklahoma	90,000	253,000	75,000	208,000
Texas	306,000	661,000	255,000	551,000
Western States:				
Montana	3,299,000	5,197,000	2,749,000	4,331,000
Idaho	1,394,000	3,080,000	1,162,000	2,567,000
Wyoming	927,000	1,566,000	773,000	1,305,000
Colorado	1,010,000	2,036,000	842,000	1,697,000
Arizona	1,071,000	2,038,000	893,000	1,698,000
New Mexico	192,000	527,000	160,000	439,000
Utah	892,000	1,581,000	743,000	1,318,000
Nevada	206,000	331,000	172,000	276,000
Washington	5,918,000	10,376,000	4,932,000	8,647,000
Oregon	3,777,000	7,935,000	3,148,000	6,613,000
California	4,109,000	9,382,000	3,424,000	7,818.000
SUMMARY.				
North Atlantic States	2,670,000	4,910,000	2,225,000[1]	4,092,000[1]
South Atlantic States	2,813,000	5,107,000	2,344,000[1]	4,256,000[1]
North Central States	2,079,000	4,270,000	1,733,000[1]	3,558,000[1]
South Central States	1,726,000	3,342,000	1,438,000[1]	2,785,000[1]
Western States	22,705,000	44,049,000	18,996,000[1]	36,707,000[1]

[1] This total is arrived at by deducting one-sixth from the total of the 90 per cent efficiency basis; the sum of the items column exceeds the total by 7,900 horsepower, due to the fractional gains in making one-sixth reduction for the individual States.

The same authority reported 4,016,127 horse-power in actual use, of which 2,961,549 H. P. were classed as commercial and 1,054,578 H. P. as manufacturing. Only those plants of 1,000 H. P. or more capacity are taken into account. In the same report, it was estimated that the aggregate capacity of plants of less than 1,000 H. P. was about 2,000,000 H. P., making a total, in round figures, of 6,000,000 H. P. developed horse-power for the United States.

M. O. Leighton[1] estimates (year 1915) that the total water power

[1] The Water Power Resources of the United States. Proceedings of the Second Pan American Scientific Congress. Washington 1916.

development in the country, excluding plants of less than 1,000 H. P. capacity is in round numbers 4,600,000 H. P. The data, classified by States, is shown in Table LXXIV.

Table LXXIV. *Capacity of developed water powers in the United States.*

	Capacity, horsepower		Capacity, horsepower
Alabama	86,450	Nevada	10,660
Arizona	27,890	New Hampshire	120,108
Arkansas	0	New Jersey	7,200
California	601,425	New Mexico	0
Colorado	69,965	New York	715,500
Connecticut	60,250	North Carolina	91,000
Delaware	0	North Dakota	0
District of Columbia	0	Ohio	4,025
Florida	5,000	Oklahoma	0
Georgia	173,427	Oregon	76,120
Idaho	116,415	Pennsylvania	169,632
Illinois	65,450	Rhode Island	0
Indiana	14,370	South Carolina	200,497
Iowa	121,400	South Dakota	0
Kansas	6,800	Tennessee	62,000
Kentucky	0	Texas	0
Louisiana	0	Utah	64,950
Maine	310,450	Vermont	97,545
Maryland	0	Virginia	51,320
Massachusetts	182,220	Washington	328,530
Michigan	133,102	West Virginia	21,400
Minnesota	183,015	Wisconsin	205,492
Mississippi	0	Wyoming	0
Missouri	18,000		
Montana	238,400	Total	4,640,208
Nebraska	0		

219. The Work of the United States Geological Survey.

The Geological Survey, in one of its branches, makes systematic stream measurements throughout the United States for the purpose of obtaining precise data regarding all its water resources. The hydrographic operations of this Corps may be divided into three distinct classes[1]:

1st: The measurement of surface streams;

2nd: Surveys of sites for, and estimates of capacity and cost of reservoirs;

3rd: The study of the quantity and movement of underground waters. The work of ascertaining the flow of a stream consists in:

a) observing and recording daily the height of the water;
b) measuring the quantity of flow at different heights;
c) computing the probable flow at each height or stage of the stream.

The Geological Department has issued the following regulations[2], the work of compilation being mainly based on the records and reports of its numerous hydrographers.

[1] Water supply and Irrigation paper No. 56 of the U. S. Geological Survey.
[2] Hydrographic Manual of the U. S. Geological Survey.

General statement: The regulations which govern all work done by the U. S. Geological Survey require that all original records of data collected shall be filed for public reference at the office of the Survey, in Washington. These records should be so clear and full in all respects that they will be intelligible at any time to engineers and other persons who may desire to consult them.

Duties of district hydrographers and engineers. In order to systematize stream measurements, the country has been divided into districts, each of which has been placed under the supervision of a district hydrographer or engineer. It is the duty of this officer to superintend all stream measurements in his territory, and he is held responsible to the chief engineer for the character of the work done by the men under him. He should have immediate supervision of all the more important features of the work which cannot be entrusted to his assistants. The stream-gauging work, which is carried on in connection with the investigation of irrigation projects, should be under the direct charge of a man designated by the district engineer, through whom he shall report. This man should be held responsible for the proper execution of all stream measurement work, and all data concerning such work must be transmitted through him.

Care in keeping records. As the work of the Government is for the general public, it is subject to more severe criticism than the work of private engineers. It is therefore essential that great care be taken in the work and that every possible excuse for criticism be eliminated. All necessary notes should be taken in the field, and, as a special precaution, at the end of each day's work notebooks should be reviewed and amplified while the facts and conditions are still fresh in the mind of the hydrographer or engineer. Careful cross references should be made, and sketches to show the relative location and details of special features should be freely used.

Computing field notes. Original computation of all field notes should be made when possible, in the field, or the notes should be computed at as early a date as possible, and in no case should the hydrographer allow large quantities of undigested data to accumulate.

Checking of records. All records and notebooks should be examined and checked before they are transmitted to the Washington office. In order to fix the responsibility for errors each book or sheet should be stamped and initialed "Computed by———", "Computations checked by ———", "Examined by———". All records are carefully examined at the Washington office, and if incomplete they will be returned to the district engineer or hydrographer for correction and amplification.

Duplicate records. The district hydrographer may cause to be prepared such duplicate copies of original notes as may be necessary for reference in his office. Too much duplication should be avoided. The original records may be loaned at any time from the files of the Washington office.

Transmission of data to Washington office: All data, correspondence and other material from each hydrographic district should be transmitted through the district hydrographer or engineer to the Washington office. This material should be initialed by the district hydrographer or by some one designated by him. With the exception of single gaugeheight and discharge measurement cards, mailed as postal cards, all data sent should

be accompanied by a letter stating the amount and character of material furnished. No data will be accepted by the Washington office which is not O.K.'d by the district hydrographer or by some one designated by him.

Standard forms. As far as possible, standard forms furnished by the survey should be used in transmitting data. Such information as can not be written on these forms and cards may be transmitted on the separate sheets of paper furnished by the Survey. These sheets should be of the standard size, 8 by 10 inches, as this is the size for which the file boxes of the Washington office are designed. Pieces of paper of odd sizes should not be used either for drawings or for reports. Sketches and maps accompanying reports should, whenever possible, either be on paper of standard size, or should be folded to conform with this size in order that they may be filed with the written reports.

The following is a list of the forms which have been adopted for general use in hydrographic work. These are intended both to minimize and simplify the work. Whenever forms become too numerous they lose their usefulness. Every effort has therefore, been made to reduce the number of forms to a minimum and to prepare these in the simplest manner possible.

List of standard forms.

Gange height forms.

Form 9—176.—Gauge-height cards, for transmitting observer's daily gauge reading to the resident and district hydrographer, to be forwarded to the Washington office.

Form 9—175.—Observer's gauge height books, for recording daily gauge heights at stations.

Form 9—212.—Daily gauge height form, for tabulating daily gauge heights arranged by months, with sufficient space for one year's records.

Discharge measurement forms.

Form 9—198.—Current meter notebooks for recording notes of discharge measurements, vertical velocity curves, and other meter work.

Form 9—221.—Card for reporting discharge measurements to the Washington office.

Form 9—207.—For tabulating discharge measurements for regular stations with spaces for all necessary data.

Form 9—223.—For tabulating miscellaneous discharge measurements.

Form 9—244.—For describing the method of and for reporting miscellaneous discharge measurements made by floats, to be used in regular work.

Computation forms.

Form 9—192.—For tabulating daily gauge heights and their corresponding discharges, with spaces for totals, monthly means, and run offs. This form provides for one year's records, arranged by months.

Form 9—210.—For tabulating station rating tables.

Form 9—206.—For tabulating meter rating tables.

Report forms.

Form 9—197.—Description of river stations to be used for describing both temporary and permanent stations.

Form 9—237.—For reporting the conditions at river stations and giving the cost of maintenance.

Form 9—213.—For reporting changes and other information in regard to river stations.

Form 9—171.—Monthly report on river stations.

Form 9—245.—Monthly report of services, giving daily employment.

Vouchers and miscellaneous forms.

Form 9—230.—A blank form ruled for miscellaneous tabulation.

Form 9—904.—Cross section notebook.

Form 9—903.—Level notebook.

Form 9—253.—Index blanks for notebooks.

Form 9—019a—Subvouchers.

Form 9—009.—Service vouchers for field men.

Form 9—005.—Field vouchers.

Form 9—015.—Traveling expense vouchers.

Form 9—918.—Expense account notebook.

Aside from these forms, which are in general use, there are several forms which are used in the Washington office in special work, hydrographers desiring forms for any purpose will kindly forward a description of what they want, and they will, as far as possible, be supplied.

Instructions for use of forms.

Observers' gauge height books (form 9—175). In the observers' gauge height books are recorded the daily stage of rivers, together with miscellaneous information, such as data in regard to floods, ice, weather conditions, etc. The observer should be instructed and trained to keep the books in as legible condition as possible, and to insure this the hydrographer should inspect the gauge height book at each visit to the station. At the end of each year the observer should be furnished with a new book and the old one should be returned to the district hydrographer's office, where it should be examined, checked, and forwarded to the Washington office for filing.

Gauge-height cards (form 9—176).—Gauge height cards should be made out at the end of each week by the local observer and mailed to the district hydrographers office, where they should be carefully examined. In this examination all missing data, such as the date, name of station, river, State etc., should be supplied. The gauge heights should be compared with the gauge heights for previous weeks to see that no unaccountable changes due to error in reading the gauge, have been reported, and the column for means should be filled in. A copy of the data on the cards should be made for the office of the district hydrographer and the originals should then be forwarded to the Washington office.

Current meter note books (form 9—198). In these books are recorded the field notes of discharge measurements and vertical velocity curves. In all cases the original data should be written in the book at the time of the measurement. In no case should it be copied from rough notes in other

books. Explanatory notes, cross references, and sketches should be added at the end of each day's work. The blank spaces at the head of each page are to be filled in; also the columns for the original data. In the column headed "Depth of observation" the exact depth for each observation should be recorded; "at 0.6 depth" is not sufficient; the position of the initial point, edges of water, edges of dead water, and number of channels should be stated; also the direction and force of wind, roughness of the bed, and any other factors on which the accuracy of the work depends. A statement of the reliability of the results should be made in the notes. If the measurement is not made at the station, the approximate distance and a brief statement of the conditions at the places of measurement should be made. The velocity at the water's edge and at the edge of piers, shoals etc., should be estimated and recorded. The cross section pages at the end of the book are to be used for sketches and additional explanatory notes. Cross reference notes should be made referring to these pages. A sample page of current meter notes with the computations for same has been shown in the chapter on stream measurements, page —.

Discharge measurement cards (form 9—221). — These are to be made out by the hydrographer and sent to the local office when the measurements are computed. Care should be taken to fill out all portions of the card. Special attention is called to the remarks and the portion for "gauge verified and found". Measurement is not complete without this data. Either "at permanent station", "temporary station" or "miscellaneous measurements" should be crossed out so that the card shows at once to which class of measurements it belongs. All cards should be carefully examined before they are sent to the Washington office.

Form 9—213. In order that the office may keep a complete and systematic record of all the changes and additions at river stations such as new gauges, bench marks, changes in the river bed, equipment, etc., this form has been prepared. It is to be used in sending to the office any data in regard to the regular river stations which affect the measurements or the station in any way. These forms are filed with the regular data for the station, and are necessary in order that the Washington office may thoroughly understand the conditions under which the station has been maintained and the measurements made.

Description of river stations (form 9—197). Special care should be taken in filling out the description of stations. The description should be so clear that, with it, a stranger could go to the locality and be able to continue the work of the station. The name of the station should be carefully chosen, and it should definitely locate the station with respect to some of the prominent features in the vicinity, as above or below some creek or tributary, at a certain bridge, near a certain township line, etc. If there has ever been another station on the same stream in the vicinity of the new station, the relative location of these two stations should be stated, and, if possible, the relation of the zeros of the two gauges. In order to facilitate the preparation of rating curves it is desirable that care be taken in preparing a sketch of the cross section of the stream. This, with the other sketches, may be made on cross section paper and attached to the description. It is also desirable to cut from a topographic sheet or some other map a section which will show the location of the station.

Indexing notebooks. All notebooks should be carefully indexed on the regulation blank 9—253 before they are forwarded to the Washington office. The index slip should be left loose in the book, in order that the file clerk may more readily check the contents. It will then be pasted on the cover. The following shows a sample of the indexing:

9—253

Department of the Interior.

United States Geological Survey.

Hydrographic Records.

Kind of notes, *Current-Meter.*
File No., *5399.*
Date received, *February 1, 1904.*
Hydrographers, *Chandler, E. F., and Richards, R.*

Stream.	Locality.	State.	Date.	Page.
			1903.	
Heart	*Church's ranch (Richardton)*	*N. Dak.*	*S. 5*	*8*
Little Missouri	*Medora*	„	„	*7*
Mouse	*Minot*	„	*S. 25, O. 5*	*9, 11—12*
Pembina	*Neche*	„	*S. 28*	*10*
Red Lake	*Crookston*	*Minn.*	*O. 12*	*13—14*
„ „	*E. Grand Forks*	„	*Ag. 4*	*6*
Red River of the North . .	*Grand Forks*	*N. Dak.*	*Ag. 1, 4*	*1—2, 4*
„ „ „ „ „ . .	„ „ *(above forks)*	„	*Ag. 1, 4*	*3, 5*

Note.—Rivers to be arranged in alphabetical order. The same entries occurring several times should be entered but once and the various pages given.

If references are to consecutive pages, use dash, e. g., 9—12; if not to consecutive pages, use comma, e. g., 1, 3, 5, 8.

Enter under the noun, except where the adjective is a recognized part of the name, thus, Platte, South fork; but North Platte, Little Missouri.

Kinds of reports. Reports in general may be divided into two classes: First, the regular monthly reports of work accomplished; second: special reports transmitting data or information to the office. The regular monthly reports are intended to give the hydrographer an idea of the work that is being done and the progress that is being made. The reports containing data go into the office files as permanent records and are used in making up the publications. An effort should be made to maintain this division.

Use of maps and sketches. In all reports, descriptions of stations etc., maps and sketches should be freely used to show the location of river stations, the sections of the country visited, and other details. In case topographic sheets have been prepared for the region under investigation the hydrographer should supply himself with copies of those sheets and make use of such portions of them as cover the territory examined. Upon these maps special features should be shown. In the absence of topographic sheets, land office and other maps should be used.

Report maps. Each district hydrographer should prepare a map of his territory from the best maps available. This map should show with their names, the principal streams and other features on which work will be carried on. He should divide his territory into its principal drainage areas and indicate them on the map. When this division has been decided upon, it should be carefully adhered to, so that the measurements and other work can be referred to drainage areas. These maps may be prepared on

tracing cloth, and from the Washington office can make a paper negative, from which as many copies as desired can be supplied. This map is desired for use in reports. With each report a map should be transmitted showing the localities that have been investigated, which should be referred to by number, so that the chief engineer can at a glance see what is being done.

Monthly reports: A monthly report of the district hydrographer is to be made up of:

a) A brief written report stating what was done during the month and the condition of the work.

b) Form 9—171, on which shall be given an alphabetical list of the river stations in each State in his district. Stations in different States should not be shown on th e same sheet, as these sheets are filed by States, so that separate sheets should be prepared for separate States. In the column of remarks, such notes as "new bench mark established", "station discontinued" (with date), "station established" (with date), "temporarily closed", etc., are to be given. Names of the stations which are temporarily closed should be kept on the list until they are discontinued.

c) Form 9—245 is designed for use by each man under the district hydrographer. On this form should be briefly stated what the employee did on each day during the month. Such general statements as clerical work, field work, etc., are not sufficient. In preparing the monthly report it is suggested that the district hydrographer should have each man under him fill out form 9—245 and form 9—171 for the stations which he has visited. From these two forms the monthly report can be taken.

Each field assistant and a distant hydrographer, when on field trips, should forward to the district office immediately after visiting a station or after completing a reconnaissance or any piece of work, the general results of the work done, and such detailed information as is necessary in making up the monthly report for that district. This enables the district hydrographer to complete his report without waiting for the return of an assistant who is making an extended trip.

Resident hydrographer's monthly report. The following is a sample form for monthly report of resident hydrographer:

Altanta, Ga., October 31, 1903.

Mr. F. H. Newell, Chief Engineer.

Dear Sir: The following is a report of the hydrographic work carried on in the States of Alabama, Georgia, and Tennessee during the month of October, 1903:

The following men were engaged: Messrs. R. R. Hall, J. M. Giles, and O. P. Hall of the regular force, and Messrs. W. G. Green and B. S. Drane of the temporary force. Their duties were as follows:

M. R. Hall had general supervision of the work, spent ten days in the office in general correspondence, preparation of descriptions, computation on meter notes, and miscellaneous duties. The remainder of the month was spent in making a reconnaissance of Flint River and its tributaries.

J. M. Giles spent the whole month in the field, making meter measurements, repairing gauges and establishing bench marks.

O. P. Hall spent the whole month in the office, plotting discharge measurements and making rating tables.

W. G. Green was employed for ten days during month in making low water measurements.

R. S. Drane was employed for twenty days during the month, ten of which were spent in the field assisting in the reconnaissance of Flint River; the remainder of the time was spent in the office working on the report of the work.

The accompanying lists show the distribution of the gauging stations in the various States; also the work done at each station. On the index map is indicated the location of the stations and of the localities investigated.

The accompanying reports and forms give a portion of the data mentioned in this report. The remaining data will follow in a few days.

Very Respectfully

M. R. Hall

Hydrographer.

9—171.

Department of the Interior.

United States Geological Survey.

Division of Hydrography.

Atlanta, Ga., October 31, 1903.

Mr. *F. H. Newell,*

In charge of Division of Hydrography.

Sir: The following is a brief statement of the hydrographic work performed during the month of *November,* 1903, under the direction of *M. R. Hall;*

River Stations Maintained, Established, or Discontinued.

Stream.	Station.	State.	No. of discharge measurements.	Remarks.
Alabama River . . .	*Montgomery* . . .	*Alabama*	0	*New B. M. established Nov. 15, 1903.*
Alabama River . . .	*Selma*	"	1	
Black Warrior R . . .	*Cordova*	"	0	
Black Warrior R . . .	*Tuscaloosa*	"	0	*Discontinued Nov. 30, 1903.*
Cahaba River	*Centerville*	"	2	
Choccolocco Cr	*Jenifer*	"	1	
Coosa River	*Riverside*	"	1	*Gauge washed out, Nov. 10; replaced Nov. 20.*

Under "Remarks" state date of establishment or discontinuance, changes in gauge, interruptions in observations, etc. A brief report covering any matters of interest occurring during the month, and form 9—245, should usually accompany this form. These make up the monthly reports of the resident hydrographers.

Furnishing information to the public. All data collected by Survey employees should be submitted to the Director through the hydrographer in charge before it is given to the public. In general requests for data which are ready for distribution should be addressed to the Director or Hydrographer in Charge, in Washington. When, however, requests are received by the district hydrographers or engineers, they may furnish the information

direct, if in so doing they are satisfied that the Survey will be in no way involved. Copies of all such requests and their answers should be sent to the Hydrographer in Charge for the Washington letter files. When the district hydrographer receives requests for data and he has some doubt concerning the advisability of complying with the request he should prepare a letter containing the information desired, for the signature of the hydrographer in charge, who will decide whether the information should be given out. A carbon copy of the reply, with the answer and the original request, should accompany the original request, so that a record of the data furnished may be kept in the Washington office.

Miscellaneous information. It is expected that each district hydrographer and his assistants shall be on the lookout for any valuable information on the hydrography of the section in which they happen to be. This information should be systematically collected and carefully filed and indexed so as to be easily consulted. It is suggested that each employee supply himself with a small pocket case to hold 3 by 5 inch index cards. When in the field, items of importance may be noted on these cards, one subject to a card, and these, when filed in the office, will be a valuable source of information.

Publications containing progress reports of stream measurements. The progress report of stream measurements for each year contains the results of the year's field work, including all original.

9—245.

Department of the Interior.

United States Geological Survey.

Hydrographic branch.

Report of services rendered by *J. M. Giles* during month of *November*, 190*3*.

[To accompany vouchers form 9—009 or subvouchers 9—019, giving briefly the character of work performed each day.]

1. *Sunday.*
2. *Plotting discharge measurements, checking gauge-height cards.*
3. *General correspondence and report on reconnaissance of Flint River.*
4. *Measured discharge of Alabama River at Salem, en route to Montgomery.*
5. *Measured discharge of Alabama River at Montgomery.*
6. *Computing discharge measurements and general correspondence.*
7. *Making up monthly reports.*

I certify that the services were rendered as above stated. *J. M. Giles.*

Approved by *M. R. Hall.*

Reports on reconnaissance, surveys, investigations, etc.—As soon as practicable after the completion of the reconnaissance work, surveys, investigations, etc., a comprehensive report should be transmitted, stating concisely the information collected and the deductions which have been made, based upon the investigations. It is not desirable to accumulate data for a semi-annual or yearly report. The material should be separated, as far as possible, into independent unit reports, and these should be transmitted as the work progresses.

Reporting new river stations. When a river station is established a statement should be transmitted, together with a description of the station, on form 9—197, stating the general reasons for establishing the station and the conditions which led to its establishment at the particular site selected.

Authority for carrying on the work. At the beginning of each season or before starting work which involves a considerable expenditure of money, the hydrographer or engineer in charge should submit for the approval of the chief engineer a brief outline of the work which it is proposed to carry on. This should state the purpose for which the data are to be collected, the cost, and other details which will give the chief engineer an idea of what it is proposed to do. Approval of the plan is to be considered authority for carrying on such investigations as may be deemed necessary to accomplish data such as gauge heights and discharge measurements, together with the office computations.

220. Legislation of Water Power in the United States.

Owing to the passage of the Federal Water Power Act, the handling of water power applications and the granting of permits has been entirely changed.

The Water Power Use Book, issued in 1911 by the Department of Agriculture, must now be considered as entirely out of date.

The reader will find in the following lines the most important points of the present legislation: method of application and provisions of the stipulations and permit.

The Federal Water Power Act[1] to create a Federal Power Commission is worded as follows:

Be it enacted by the Senate and House of Representatives of the United States of America in Congress assembled, That hereafter no permit, license, lease or authorization for dams, conduits, reservoirs, power houses, transmission lines, or other works for storage or carriage of water, or for the development, transmission, or utilization of power, within the limits as now constituted of any national park or national monument shall be granted or made without specific authority of Congress, and so much of the Act of Congress approved June 10, 1920, entitled "An Act to create a Federal Power Commission; to provide for the improvement of navigation; the development of water power; the use of the public lands in relation thereto; and to repeal section 18 of the River and Harbor Appropriation Act, approved August 8, 1917, and for other purposes," approved June 10, 1920, as authorizes licensing such uses of existing national parks and national monuments by the Federal Power Commission is hereby repealed.

Approved March 3, 1921.

The commission has jurisdiction over all projects involving the construction, operation, and maintenance of dams, water conduits, reservoirs, power houses, transmission lines, or other project works for the development, transmission, and utilization of power which affect navigable waters and other waters over which Congress has conditional jurisdiction and public lands or reservations.

Permits or valid rights of way granted prior to and existing on June 10, 1920, remain in force unaffected by the provisions of the Federal Water Power Act, but any person, association, corporation, State, or municipality holding or possessing any such permit, right of way, or authority may apply for a license under said act, and thereupon the provisions of the act will apply to the applicant as a licensee thereunder.

[1] Amendatory Act, approved March 3rd 1921.

The following regulations[1] will be of interest to the engineer and guide him as to the steps that must be taken in order to comply with the Government requirements.

Regulation 2.—Applications—General Requirements.

Section 1. Applications for preliminary permits or licenses may be filed either with the commission at Washington, D. C., or with such office, agent, or agency of the commission as may from time to time be established or authorized to receive applications on behalf of the commission.

Sec. 2. Applications filed with the Departments of War, Interior, or Agriculture, prior to approval of the act, under the provisions of then existing law and accepted as complete by the department with which filed will upon written request of the applicant be received by the commission as an application under the act.

Sec. 3. The date of filing of applications shall be the date of receipt of the latest communication containing information essential for action by the commission on the application.

Sec. 4. All applications for preliminary permits or for licenses shall include:[2]

A. Information relative to the applicant's qualifications to make application as follows:

(1) If the applicant is a corporation:

(*a*) A copy of its charter or articles of incorporation duly certified by the secretary of state of the State where organized, or other officer having legal custody of a record of incorporations (one copy only, marked "Exhibit A").

(*b*) Execute Form 1 in triplicate and mark it "Exhibit B."

(2) If the applicant is a State:

A copy of the law or laws, or reference thereto, under authority of which the application is made, duly certified by the governor or secretary of state of the State, under seal, and two uncertified copies marked "Exhibit B."

(3) If the applicant is a municipality:

(*a*) A copy of its charter or other organization papers duly certified by the secretary of state of the State in which it is located or other officer having legal custody of the record of municipal incorporations (one copy only, marked "Exhibit A").

(*b*) A certified copy, or reference thereto, of the State law or laws authorizing the operations contemplated by the application and two uncertified copies marked "Exhibit B."

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[1] Federal Power Commission. Orders No. 9. Rules and Regulations concerning the improvement of navigation; the development, transmission, distribution and utilization of water power; and the use of lands and other property of the United States in relation thereto. Issued under the Act of June 10, 1920 (41 Stat., 1063). Approved on and effective from February 28, 1921.

[2] If any applicant shall have filed with the commission any of the papers required by this regulation, a specific reference to such filing, accompanied by duly certified statements of changes thereafter made, thereby bringing up to date the information of the previous filing, will be accepted as fulfilling the requirements with respect to such papers.

(4) If the applicant is an association of citizens;

(*a*) Each member shall make affidavit of citizenship. (See application Forms 3 and 4.) Associations shall submit their articles of association. If there be none, the fact shall be stated over the signature of each member of the association (one copy only, marked "Exhibit A").

(*b*) A complete list of members must be given in an affidavit by one of them (in triplicate, marked "Exhibit B").

(5) If the applicant is an individual:

An affidavit by the applicant that he is a citizen of the United States. (See application Forms 3 and 4.)

B. General data with respect to the project or projects applied for, to be shown on the prescribed application Form 3 or 4, paragraphs 2 to 6, inclusive.

C. Maps and drawings as required by Regulation 3, 4, or 5, as the case may be. All maps and drawings:

(1) Shall be on tracing linen cut to uniform size, not smaller than 24 inches by 36 inches and not larger than 28 inches by 40 inches, the latter size being preferred, except that lithographed official maps issued by Federal or State agencies may be used for general maps of the project and, when su used, one copy for permanent record shall be mounted on linen.

(2) Shall have a clear border of one-half inch on three sides, and of $2\frac{1}{2}$ inches on one of the shorter sides, which shall be the left-hand border of the map.

(3) Shall have a numerical scale and a graphical scale, the latter not less than 6 inches in length.

(4) Shall, if a map, have true and magnetic meridians indicated thereon.

(5) Shall have a space 4 inches by 7 inches in the lower right corner, the upper half of which shall bear the title, scale, etc., and the lower half of which shall be left clear.

(6) Shall, if a map including public lands, show location of all official public-land survey lines crossing the project area. If on unsurveyed public lands or reservations, the protractions of township and section lines shall be shown, such protractions whenever available to be those recognized by the agency of the United States having jurisdiction over the lands.

(7) Shall be rolled, not folded, for mailing, etc.

Sec. 5. Applicants may be required upon request of the commission to furnish maps, plans, or other data in addition to the requirements specifically set forth in the regulations, if in the judgment of the commission such additional data are desirable for a full understanding of the project or projects or for the purpose of enabling the commission to review the design thereof, either in whole or in part.

Sec. 6. The names of applicants, the information contained in the application forms (Forms 3 and 4), and general project maps will be available for general public information. Other information concerning applications will not be made public except upon a showing satisfactory

to the commission of the public necessity or desirability therefor, or with the consent of the applicant. At public hearings upon any application the commission may require the presentation of such information respecting the application as in its judgment the proper conduct of the hearing or the public interests requires.

Regulation 3.—Applications for preliminary permits.

All applications for preliminary permits shall be submitted in triplicate on Form 3, and shall be accompanied by the items specified below, each of which shall be dated and signed by the applicant in the form prescribed in Form 2 and designated "Exhibit A," "Exhibit B," etc., as indicated.

Section 1. *Exhibit A.*—Certified copy of charter, articles of incorporation or other organization papers (one copy only). (See Regulation 2, sec. 4.)

Sec. 2. *Exhibit B.*—Other evidence that applicant is qualified to apply for and receive a permit (in triplicate). (See Regulation 2, sec. 4.)

Sec. 3. *Exhibit C.*—Statement of nature and amount of data available, such as surveys, maps, plans, stream measurements, foundation explorations, etc., and of work already performed, including preliminary construction, such as clearing, road building, etc. (in triplicate).

Sec. 4. *Exhibit D.*—Nature and amount of work proposed to be performed under preliminary permit, such as surveying, preparation of maps and plans, measurement of streams, explorations of foundations, or preliminary construction, and estimate of cost thereof (in triplicate).

Sec. 5. *Exhibit E.*—Statement of nature, extent, and ownership of water rights and of ownership of lands which applicant contemplates using in the development of each project. Also statements of applicant's plans with reference to perfecting water rights and to acquiring lands (in triplicate).

Sec. 6. *Exhibit F.*—Statement whether project involves the use of a Government dam or other Government structure; and if so, to what extent (in triplicate).

Sec. 7. *Exhibit G.*—Such data as the applicant desires to supply as to his ability to finance the preliminary work as well as the project or projects applied for (in triplicate). The commission may decline to approve the application unless satisfied that the applicant will be able to finance the preliminary work and the project.

Sec. 8. *Exhibit H.*—A general map showing the nature, principal features, and location of the project with reference to some well-known town or stream. On this map shall be placed a line indicating the approximate project boundary (exclusive of transmission lines) and the approximate distance and bearing of the upper and lower limits thereof from some natural object or permanent monument that can be readily found and recognized from a description thereof noted on the map. If on land covered by official public-land survey, the distances and bearings shall be from the nearest existing corner of the public survey. (One tracing and two blue prints, or lithographed official map with project shown thereon, one copy mounted on linen and four copies not so mounted.) (For size, etc., of map, see Regulation 2, sec. 4, par. C.)

Sec. 9. *Exhibit I* (to be submitted only if public lands or reservations are affected).—A map showing the project area and the location of the principal features of the project. The map shall show as the project boundary the subdivision line of the public-land survey lying next outside the project area, unless the applicant can define it more closely, in which case the boundary shall be so defined.

On the map shall be indicated separately lands patented, lands entered or otherwise embraced in any unperfected claim under the public-land laws, unreserved public lands, and lands of each and every reservation affected. The map shall be on a scale to present this information effectively.

If so desired, a single map may be submitted for Exhibits H and I, provided it shows all the information required for both, and, in that case, one tracing and five blue prints shall be furnished. If separate maps are used, one tracing and three blue prints shall be furnished. (For size, etc., of map, see Regulation 2, sec. 4, par. C.)

Sec. 10. *Exhibit J.*—Such additional exhibits as applicant may consider pertinent to submit.

Regulation 4.—Applications for licenses—major projects.

All applications for licenses, and for additions or betterments to projects already under license (except for projects of not more than 100-horsepower capacity), shall be submitted in triplicate on Form 4, and shall be accompanied by the items specified below, each of which shall be dated and signed by the applicant in the form prescribed in Form 2 and designated "Exhibit A," "Exhibit B", etc., respectively, as indicated.

Section 1. *Exhibit A.*—Certified copy of charter, articles of incorporation, or other organization papers (one copy only). (See Regulation 2, sec. 4.) If already furnished for preliminary permit, reference thereto will suffice.

Sec. 2. *Exhibit B.*—Other evidence that applicant is qualified to proceed with the project applied for (in triplicate). (See Regulation 2, sec. 4.) If already furnished for preliminary permit, reference thereto will suffice.

Sec. 3. *Exhibit C.*—Field notes, or a description by metes and bounds, of the entire final location survey of the project boundaries and of transmission lines. Field notes need not be submitted as a separate exhibit when they are noted on tracing filed with application (in triplicate).

Sec. 4. *Exhibit D.*—Evidence that the applicant has complied with the requirements of the laws of the State or States within which the project or projects will be located with respect to bed and banks and with respect to the right to engage in the business of developing, transmitting, and distributing power, and in any other business necessary to effect the purposes of the license applied for. This evidence shall be accompanied by a statement of the steps that have been taken and the steps that remain to be taken to acquire franchise of other rights from counties and municipalities before the project or projects can be completed and put into operation (in triplicate).

Sec. 5. *Exhibit E.*—Statement of the nature, extent, and ownership of the water rights and of ownership of lands which the applicant pro-

poses to use in the development of the project or projects applied for, together with satisfactory evidence that the applicant has proceeded as far as practicable in perfecting its rights to use sufficient water for proper operation of the project works. A certified statement from the proper State agency setting forth the extent and validity of the applicant's water rights shall be filed if practicable. Whenever the approval or permission of one or more State agencies is required by State law as a condition precedent to the applicant's right to take or use water for the operation of the project works, duly certified, evidence of such approval or permission or a showing of cause why such evidence can not be reasonably submitted shall be filed. Statement of applicant's plans for acquiring lands owned by other parties and which are essential for carrying out the project. If statement submitted with application for preliminary permit gives above information, a reference thereto will suffice (in triplicate). When State certificate is involved, one certified copy and two uncertified copies will suffice.

Sec. 6. *Exhibit F.*—Statement whether project involves the use of a Government dam or other Government structure; and if so, a detailed description of the proposed use. Information relative to such structure which may be of record in any branch of the United States Government at Washington need not be furnished if appropriate reference thereto is given (in triplicate).

Sec. 7. *Exhibit G.*—Statement showing the financial ability of the applicant to carry out the project or projects applied for (in triplicate). If adequate statement has been furnished with application for preliminary permit, reference thereto will suffice.

Sec. 8. *Exhibit H.*—Statement of the effect of the proposed operation of the project works on the normal flow of the stream, including a statement of the minimum flow proposed to be released during periods of low water and full exposition of the relation of any proposed ponding of the flow to the conservation and utilization in the public interest of the available water resources for the purposes of power, navigation, irrigation, reclamation, flood control, and municipal water supply (in triplicate).

Sec. 9. *Exihibit I.*—Estimate of the power capacity[1] of each project, accompanied by the complete data upon which such estimate is based, including stream flow, evaporation records, static heads, etc. A list of all present and proposed sources of power for the system of which the project or projects will be a part, including the approximate location, water power or steam power, source of water, ultimate capacity of all power plants in use or proposed to be used, and the present installed capacity of constructed plants; also connections, if any, with other power systems (in triplicate). If any of these data are of record in any branch of the

[1] The "power capacity" of a project means the continued product of—

A. The factor 0.008; which represents the horsepower at 70 per cent efficiency of one cubic foot of water per second falling through a head of 1 foot.

B. The average static head in feet; and

C. The water supply, in cubic feet per second and not in excess of the hydraulic capacity of the approved project works, estimated to be available from natural flow or from storage, or from both, for 90 per cent of the time.

United States Government at Washington, appropriate reference thereto will suffice.

Sec. 10. *Exhibit J.*—General map covering the entire project area or areas, showing on a single sheet and to an appropriate scale:

A. Principal structures and other important features.

B. The entire transmission system with which the project or projects may be connected, indicating prominently by appropriate symbol the portions for which application is made.

C. State and county lines, towns, streams, and other features that will aid in arriving at a general comprehension of the project.

D. References to the detail map, indicating by outline the portion shown on each sheet.

E. If all features can not be shown with sufficient distinctness on one sheet, two general maps may be furnished, one for structures and one for the transmission system. (Furnish one tracing and two blue prints, or one lithographed map mounted on linen and four unmounted with project works appropriately marked thereon. For size, etc., of map, see Regulation 2, sec. 4, par. C.).

Sec. 11. *Exhibit K.*—Detailed map covering entire project area or areas. Scale in general not more than 400 fect to the inch, except for such portion as will be used only for transmission or telephone lines, where scale of not more than 1 mile to the inch may be used, provided it is accompanied by field notes giving a description of the right of way for the line. Elevations shall be tied to Government bench marks whenever available and shall be referred to mean sea level, except that, in the case of projects on navigable waters having a datum accepted for local use by the United States Engineer Department, War Department, such local datum shall be used. If more than one sheet is used, the sheets shall be numbered consecutively, and each shall bear a small diagram showing the entire map and indicating the portions shown on each sheet. The map shall show with respect to each project applied for:

A. The project boundary, which shall be a line to include the area within which it is necessary to acquire land or rights of way for the project. It shall include only such lands as are or will be valuable and serviceable in the development, transmission, or distribution of power in connection with the project. The location of the project boundary may or may not follow established land lines. It shall be subject to approval of the commission, and unless satisfactory reasons are given therefor it shall not be more than 250 feet distant from exterior margins of reservoirs, water conduits, transmission lines, and appurtenances, or other project works. The project boundary shall be described in such a way that it can be legally identified, and shall be marked on the ground by existing land lines supplemented by such additional monuments as may be necessary. Such portion of the project area as will be used only for transmission lines, telephone lines, pipe lines, conduits, or canals may be described by lines of survey specifying the distances of the project boundary therefrom.

B. All project works accurately located, such as:

(1) Dams.

(2) *Reservoirs.*—Indicate the flow lines by maximum and minimum water levels and by elevation of spillway crest, and give tables or diagrams of areas and capacities for maximum and minimum water levels and for each contour line.

(3) *Water conduits.* — Indicate center line; type, i. e., flume, ditch, tunnel, pipe, etc.; and grade and elevation of bottom at each change of grade.

(4) Power houses and substations.

(5) Transmission lines and appurtenances and telephone lines.

(6) Navigation structures.

(7) Channel approaches to navigation structures indicating elevations of bottom for distances of not less than 1,000 feet above and below the structures.

C. Contour lines with contour interval of not more than 10 feet for the entire project area, except such portions as will be used only for transmission lines or telephone lines or will be included in reservoirs below the minimum elevation to which the water may be drawn down: *Provided,* That a profile of tunnel lines may be substituted for contours along such lines.

D. If on lands covered by officiel public-land survey (see Regulation 2, sec. 4, par. C (6)), show distances along land-survey lines from the nearest established corner to all intersections with the project boundary where such distances do not exceed 1 mile.

E. Reference lines to the initial points of project boundaries and of the survey (center lines) of transmission or telephone lines and to their intersections with boundaries of reservations. Such reference lines shall show distances and bearings by a line or lines that can at all times be readily retraced from an established corner of the public survey, if not more than 2 miles distant, or, if such established corner is not available, a permanent mark on a natural object or a permanent monument which can be readily found and recognized from a description thereof noted on a map or in the field notes.

F. The status as to ownership, and the land lines and area of each parcel of land within the project boundaries, designating separately lands owned by the applicant, lands for which the applicant holds rights of use and occupancy for purposes of the project, reservations (indicating separately each reservation), and public lands (indicating separately lands, full title to which remains in the United States, and lands in which the United States retains only an interest). (One tracing and two blue prints.)

Sec. 12.—*Exhibit L.*—General design drawings showing plans, elevations, and sections of all principal structures and appurtenant works or other features of each project applied for. These drawings shall be in sufficient detail and shall be accompanied by sufficient information relating to controlling factors (such as character of foundations and explorations thereof, materials, types of construction, important elevations, and water levels, etc.) to enable the commission to have a full understanding of the

project and to check safety, adequacy, and desirability in the development of the resources involved. (One tracing and two blue prints.) This section applies to:

(*a*) Dams and appurtenances, such as spillways, fishways, outlet works, etc.

(*b*) Navigation structures and approaches thereto, including locks, lock gates, and operating machinery thereof etc.

(*c*) Conduits, including forebays, intake works, surge tanks, and other pressure relief devices, etc.

(*d*) Power houses and substations.

Sec. 13. *Exhibit M*.—General descriptions and general specifications of mechanical, electrial, and transmission equipment and their appurtenances in sufficient detail to enable the commission to have a full understanding of the project or projects and to determine safety and suitability for the development and utilization of the resources involved (in triplicate).

Sec. 14. *Exhibit N*.—Estimate of the cost of developing each project, segregated by principal features, showing quantities, unit costs, etc., in sufficient detail for a full unterstanding of the elements of cost of the project (in triplicate).

Sec. 15. *Exhibit O*.—Detailed statement of the time desired for completing preliminary construction and for beginning and completing construction of the project works. If the ultimate development is to be completed and put into operation in two or more parts, the time desired for beginning and completing the construction of each part shall be given (in triplicate.)

Sec. 16. Such additional exhibits as applicant may consider pertinent to submit.

APPLICATION FORMS.

The following forms are prescribed for applications filed under the Federal Water Power Act, approved June 10, 1920 (41 Stat., 1063), in accordance with the foregoing regulations:[1]

Form. 1. — *Official statement of organization of corporation and list of officers and directors.*

[See Regulation 2, sec. 4, par. A (1) (b).]

I, ..., ...
(Name.) (Secretary or president.)

of the ...., do hereby certify that the organization of said corporation has been completed; and that the corporation is authorized to proceed with the project applied for according to the existing laws of the State of .... I further certify that the following is a true list of the principal officers and of the directors of the said corporation, with the full name and official designation of each, to wit: (Here insert the full name and official designation of each principal officer and director)

Witness my hand and the seal of the corporation this

.... day of .... 19 .

....,

....

(SEAL.)

Form 2. — *Dating and signature of applicant.*

[See Regulations 3, 4, 5.]

This map (these estimates, this copy of notice, etc.) is a part of the application for preliminary permit (license) made by the undersigned this

.... day of .... , 19 .

....

Form 3. — *Application for preliminary permit.*

[See Regulations 2 and 3.]

(Act of June 10, 1920, 41 Stat., 1063.)

([1]) ....

a {corporation / municipality} organized and existing under and by virtue of the laws of the State of .... and having its office and principal place of business at

...., in the State of ....

[1] Applications have proved to be so varied in size that is has been found impracticable to furnish printed blanks therefor. Applications should be typewritten, following the forms indicated herein.

([1]) ...., ...,

citizen of the United States, whose business address is ....

....do .... hereby make application to the Federal Power Commission for a preliminary permit for .... months, for the project described herein and approximately shown upon a certain map signed by the applicant on the .... day of .... 192 , which map is filed herewith and made a part hereof. This application is made in order that the priority of the applicant with respect to a license under the Federal Water Power Act may be maintained while securing the data and performing the acts necessary to perfect an application for such license.

2. The location of the project ..... applied for is as follows:

(*a*) In the State of ....

(*b*) In the county of ....

(*c*) On the following-named stream .... navigable and carrying commerce to the following extent:

....

....

....

....

(*d*) In the region of the following-named cities and towns:

....

....

....

3. The proposed scheme of development for the project is as follows:

....

....

....

4. The proposed use of market for the power to be developed is as follows:

....

....

....

5. The location and capacity of all power projects owned or operated by the applicant, the markets supplied thereby, and the relation thereof to the project applied for are briefly described as follows:

....

....

....

[1] Cancel words not used.

6. The following exhibitis are filed herewith and are hereby made a part of this application:

(Give each exhibit a designation and brief description, as Exhibit A, certified copy of articles of incorporation; Exhibit B, official statement of organization of corporation and list of officers on Form 1; Exhibit C, statement of nature and amount of data available, etc.)

Exhibit: ....

Exhibit: ....

Exhibit: ....

In Witness Whereof the applicant has caused its name and corporate seal to be herunto signed and affixed by

.... its ....

thereunto duly autorized, this .... day

of ...., 192...

....

By ....

....

Attest:

..........,

Secretary.

(When the application is made by an individual or association of individuals, it will be signed and sworn to before a notary public or other officer having authority to administer the oath, using the following form:)

Witness the signature .. of the applicant .. this ....

day of ...., 192...

....

....

....

....

[1] Subscribed and sworn to before me this ....

day of ...., 192...

[OFFICIAL SEAL.] ....,

Notary Public.

Form 4. — *Application for license.*

[See Regulations 2 and 4.]

(Act of June 10, 1920, 41 Stat., 1063.)

[1] ....

municipality / corporation } organized and existing under and by virtue of the laws of the State of ...., and having its office and principal place of business at ...., in the State of .... ([1]) ....

[1] This form when sworn to will serve as evidence of citizenship.

citizen .. of the United States, whose business address is ...., do .... hereby make application to the Federal Power Commission for a license for the project described herein and shown on general and detail maps signed by the applicant on the .... day of ...., 192 .., which maps are filed herewith and made a part hereof, said license to authorize the construction, operation, and maintenance of certain project works, the principal ones of which are designated as follows on said maps:

(Give the name or other designation and a brief description of the principal project works and cancel such of the four following items (*a*), (*b*), (*c*), and (*d*) as may not be applicable.)

(*a*) (Dams and reservoirs.) ....

....

....

(*b*) .... (water conduits.) ....

....

....

(*c*) .... (power houses.) ....[1]

....

....

(*d*) .... (transmission lines.) ....

....

....

2. The location of the project.... applied for is as follows:

(*a*) In the State of ....

(*b*) In the county of ....

(*c*) On the following-named stream .... navigable and carrying commerce to the following extent:

....

....

....

(*d*) In the region of the following-named cities and towns:

....

....

....

3. The proposed scheme of development for the project is as follows:

....

....

4. The proposed use or market for the power to be developed is as follows:

....

....

....

[1] Cancel words not used.

5. The location and capacity of all power projects owned or operated by the applicant, the markets supplied thereby, and the relation thereof to the project applied for, are briefly described as follows:

....

....

....

6. The following exhibits are filed herewith and are hereby made a part of this application:

(Give each exhibit a designation and brief description, as Exhibit A, certified copy of articles of incorporation; Exhibit B, official statement of organization and list of officers on Form 1; Exhibit C, statement of nature and amount of data already available, etc.)

Exhibit: ....

Exhibit: ....

Exhibit: ....

Note: Licenses under this Act shall be issued for a period not exceeding fifty years.

Index of Contents.

6 –